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Volume XIX APRIL, 1932 Number 4

SOCIETY RESEARCH

PRESIDENTIAL ADDRESS BY HARRY E. SAWTELL,* BOSTON SOCIETY OF CIVIL ENGINEERS, MARCH 16, 1932

I WISH to say a few words about the subject of Society Welfare before passing to my main subject.

It is unfortunate that a season of business and unemployment distress is in full swing, but such occurrences cannot be helped while the production of raw and finished materials remains uncontrolled, plus some other causes.

Much good, however, will come out of this season of distress, one item being a new sense of kinship of professional engineers and architects; another good result will be a real step toward solidarity of all classifications of professional engineers. Proof of the ability of engineers, in the different branches of the profession, to get together and to work for the alleviation of suffering of the few is now being demonstrated.

A great deal is said lately about the "ideal of service." Is not this work fulfilling the requirements of such a service? We are serving our brother engineers and sharing our incomes with them. We can do little more.

Individualism is one of the most valuable traits a people may possess, and a trait which accounts in large measure for the great strides made by our people since the beginning of our country. This, however, has often been accompanied by "isolation," and the professional engineer, while being extremely individualistic, has, seemingly, more than any other professional, cultivated "isolation."

If that co-operative and co-ordinated force we call cohesion can be obtained among engineers, and individualism be retained, that day will mark the time when we may expect to find engineers doing still greater work and taking their proper place, in this time of rapid advancement, and doing their work under better conditions than now exist in some communities. We have a situation among us now, which, if nurtured, will develop cohesiveness and constructive strength. I refer to our united action in assisting the distressed section of our membership and the rational relationship with professional architects.

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A prize of great value that will be captured, largely as a result of this depression, is the forced improvement of materials and methods in a multitude of instances. This will be done largely by research. In materials alone many splendid improvements have been made, in order to meet the competition for better materials at lower cost. The necessity of getting new business has caused many hitherto stagnant minds to search for better things and better methods.

Great prosperity does not encourage keenness and research as does adversity, and the world will be richer in many ways as a result of this depression. The value of research in all such emergencies has been proven, and it is mounting higher and higher.

A more exact knowledge of the things we use and come into contact with will aid us to use them with greater surety and economy. Take the case of structural steel, the more exact the knowledge of its properties became and the more reliably it was manufactured, through constant research and improvement, the greater the economy in its use became, largely through reduced safety factors.

Some one has stated that large safety factors indicate at least one thing, namely, our ignorance of physics. As this is largely true, it points clearly to the desirability of having more exact knowledge of all the things we use directly or indirectly in our designs and reports.

We must find a method of encouraging individualism in the young engineer, and we shall accomplish this if we induce him to investigate and to think for himself. In the so-called rush of the times it may seem out of place to suggest that an engineer undertake the slow work of research to clear up some obscure subject or to gather data and facts in order that an advance may be obtained in the general understanding of some physical or scientific subject, but I am suggesting just that.

A booklet issued by the Lehigh University notes two contrasting facts, which are significant. I quote it as follows:

> When the foremost statesmen of the world met in the Congress of Vienna, about a hundred years ago, to patch up the mischief done by Napoleon, a great physicist was working quietly at the University of London.
>
> The diplomats of Vienna talked as much as possible, listened when necessary,

staged a great many brilliant and expensive functions, and accomplished for the good of mankind exactly nothing.

Professor Faraday, on the other hand, mounted a copper disk between the poles of a horseshoe magnet, and by whirling the disk, set up an electric current.

From the Congress of Vienna, the world has nothing but bitterness, jealousies, and war. From Michael Faraday, the silent man of science, the world has had light for its lamps, life for its telephones, and power for its mighty machines.

Modern civilization is not built on rhetoric and graceful gestures, but on the application of scientific facts. It is well worth while to think of the men who are applying these facts. For if the world is ever to see come true the Utopia of its dreams, in which there will be no war, no famine, and no disease, the world must learn to look to the creative intelligence of its scientists and engineers.

Few are privileged to discover basic data bearing on physics, but most of us can put a drop into the bucket of advancement.

Modern life requires the use of *new* materials as well as the better use of old materials. How do we determine the true value of the great mass of new materials and forms which are thrust upon us for use? The answer is, "Research," for it is by the hard digging for more exact knowledge of their properties and physical characteristics that we gain a true idea of this value to us.

Schools should stick largely to teaching fundamentals of science and engineering, leaving the graduate to find ways of becoming acquainted with up-to-date engineering methods and materials used in the industrial and municipal fields. Industry has found that at least one thing is a necessity (whether times be good or bad), and that is "research," the constant quest for new facts and new ways of doing things.

Some time ago a speaker before this Society stated that he had derived substantial benefits from the scientific information published by the Boston Society of Civil Engineers. His case must be typical of many, and points to the desirability of continuing the publication of a steady stream of valuable and exact information, including designs, descriptions of work done, investigations and research. It is for the benefit of the coming generations, perhaps, more than for ourselves, that we should continue to publish, in increasing numbers, technical papers and results of researches.

Our fields of engineering activities are both broadening and intensifying; broadening by including the use of new materials and methods in structures, or problems, and the constantly increasing application of engineering sciences to meet the demands of modern life; and intensifying through specialization and research. Broadening our field of activities should not cause us to forget that it is along technical lines that our Society gains its greatest strength and value.

Engineers should study the trend of the times closely and develop along lines which serve the new conditions found at this time. There seems to be a more rapid change going on now than at any other time in history, and this includes means of transportation, of communication, or recreation, higher standards of living, sanitation, quality of materials, manufacture, etc.

One of the outstanding movements in this age is the astounding increase in the number of professionals who are devoting most, if not all, of their time to research work. To the great number of professional organizations which are devoting all of their time to research work in general must be added the large number employed in laboratories operated by such large corporations as the General Electric Company, Westinghouse Electric and Manufacturing Company, Bell Telephone, Dupont and others. I am told that the total number of investigators in this country runs well above 15,000. Underlying these professional and industrial laboratories is the great number of University laboratories, which are advancing the knowledge of fundamentals in science and engineering.

To show how deeply the desire to understand and improve things has grown into our blood, it is a fact that our children are experimenting, at home, with radio waves, electricity, laws of aerodynamics, and otherwise showing themselves to be far ahead of what we were at their age. Let us hope that they will know how to control some of the new things they are preparing to devise before they let them loose on mankind. If they do so, it will be doing something we have not always done.

What will be the natural result of the enormous amount of research work? We are already seeing what is probably a small part of this result, namely, the wonderful advance in knowledge of physics, new electrical appliances, new metals, and greater and better use of other materials, new machines, more economical production of power, etc. Is there any limit to which a people may go when so great an amount of well directed research is carried on?

The United States is making more rapid strides toward scientific knowledge, and, in engineering, application of such knowledge, than ever Germany did in its best pre-war days.

It is reported that Dr. Micholsen has said that "Science is lots of fun." Let me paraphrase this by saying that "Research, the servant of science, is lots of fun."

Our own Society has a great obligation to meet in carrying on its work in such a way as to satisfy its members in their desire for knowledge of better ways of doing things and for a better understanding of the materials we use.

In the immediate past there have been published in our JOURNAL certain reports requiring rather extensive labor in research and fact finding, some of which are named below:

Reports on "Boston's Foundation." J. R. Worcester. January JOURNAL, 1914 (out of print).
Report of Run-Off Committee, October, 1922.
Report of Sanitary Section on Collection and Disposal of Garbage, June, 1927.
Report of Committee on Floods, September, 1930.
Report of Subsoils Committee, September, 1931.

With one exception, these were the work of a committee.

All of them added much to the total of knowledge of the subjects covered, and enhanced the value of the Society to the community.

Another Run-Off Committee is preparing for another report to be presented in the future.

Individual papers of value on subjects requiring much research have occasionally been published, but it seems to me that the number of men of our Society giving their time to investigating worthy subjects is too small. Such work stimulates the individual and increases materially the reputation of our Society when published in our JOURNAL. The JOURNAL has a wide circulation, but it could be increased in value by publishing the results of a still greater number of original investigations and researches. No one thing comes to mind having greater significance or possibilities.

If I may be pardoned for being a little personal, let me say that the engineering organization of which I am a member has for years thought it desirable to carry on, occasionally, researches into certain things which would give data aiding them in their work. Some of the subjects investigated are:

Fractional distillation of creosote oils to see if within specification limits.
Analyses of roofing felts. This involved extraction with a solvent, microscopic fiber analysis, strength tests, and a number of other tests.
Microscopic studies of the structure of wood.
Studies of wood preservation.
Sodium fluoride experiments in soaking and steeping timber for prevention of decay.
Many soil analyses for cores of earth dams, etc.
Analyses of clays.

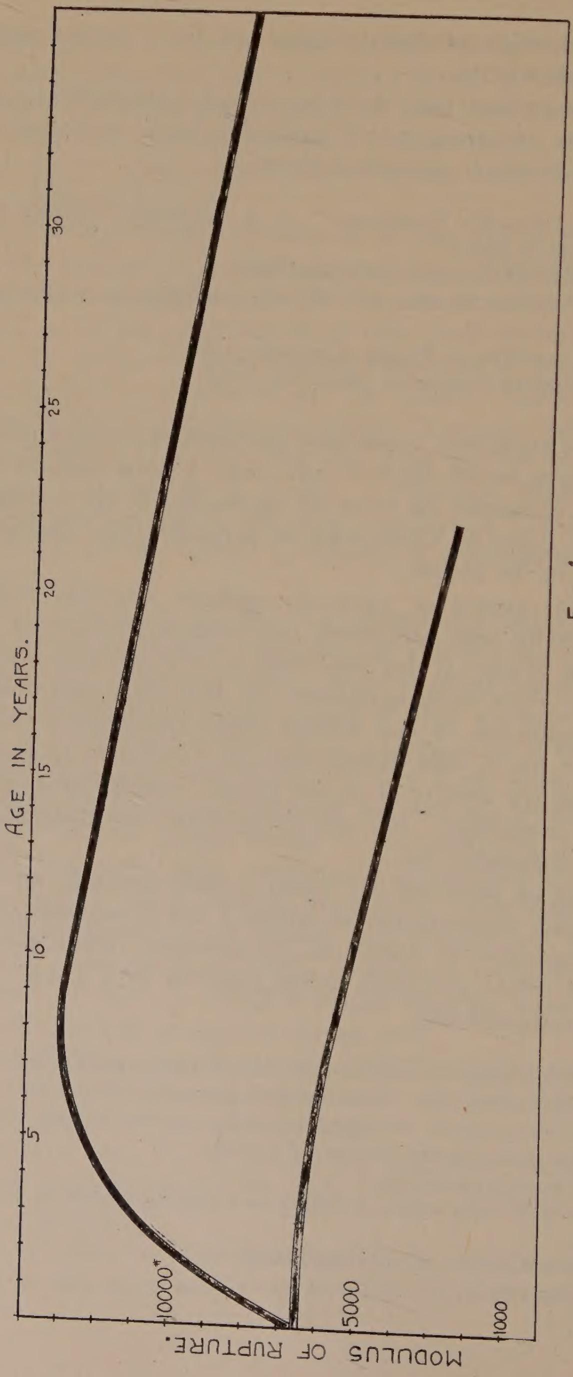

Age in Years.
5
10
15
20
25
30
Modulus of Rupture.
10000*
5000
1000

Fig 1

The above list has purposely been selected to bring out clearly that the common things with which we come into contact may be suitable subjects for research, and occasionally a subject may be of interest to engineers in general. Such appeared to be the case with the subject of "Wood Preservation. The Use of Sodium Fluoride in the Steeping Process," published in the JOURNAL of May, 1927.

The privilege has been mine this year to gain the permission of the Board of Government to appoint a committee for the purpose of scheduling subjects suitable for research either by individuals or committees. The committee, headed by Past President Frank Marston, has presented a splendid schedule of subjects, with the expressed hope that it will be an incentive to numerous members to undertake such work.

This schedule includes two subjects which have been investigated by us, and I am going to describe them briefly, in order to show the comparative ease of doing this kind of research work which, if published, may be of some help to others than the organization making them. These are, first, "The Effect of Decay on Structural Timber," and second, "The Classification of Clay for Densities."

The investigation of "The Effect of Decay on Structural Timber" was undertaken to satisfy a desire to have more exact data at hand to aid in answering the oft-repeated question which industrial plant owners ask, "How good is this floor. How much load can we apply to it safely?" These questions are not easily answered, and if one has no data to help him he is apt to play safe and condemn the floor outright. This is not always justified from an economical point of view.

Having studied rot fungus, its progress under different conditions, and its general effect upon the strength of timber, it was, perhaps, comparatively easy to make this investigation. It is not necessary to understand rot fungus from a pathological viewpoint, but it is necessary to be able to identify the fungus and its mycilium in wood and the result of its progress.

The strength-life curve, showing the probable average strength of timber placed in buildings, together with the effect of moisture content and time, is shown, in Fig. 1, to help make clear how timbers may react under different conditions.

The upper curve of Fig. 1 shows the result of putting an air-dried timber into a dry-heated building. The ultimate strength increases as the timber dries out, and remains high for a long time; then there is a decrease, due to a very slow increase in dry rot, checking, mechanical wear, overload, impact — resulting in fatigue and failure. The life under these conditions may average fifty years. Under certain favorable conditions timbers are known to be in good condition after one hundred years or more of use.

The average dense southern pine timber, when green, may have a 45 per cent moisture content and a modulus of rupture as low as 5,000 pounds per square inch. The average dense southern pine timber, *air-dried*, may have a moisture content of about 25 per cent and an ultimate strength of 6,500 pounds per square inch.

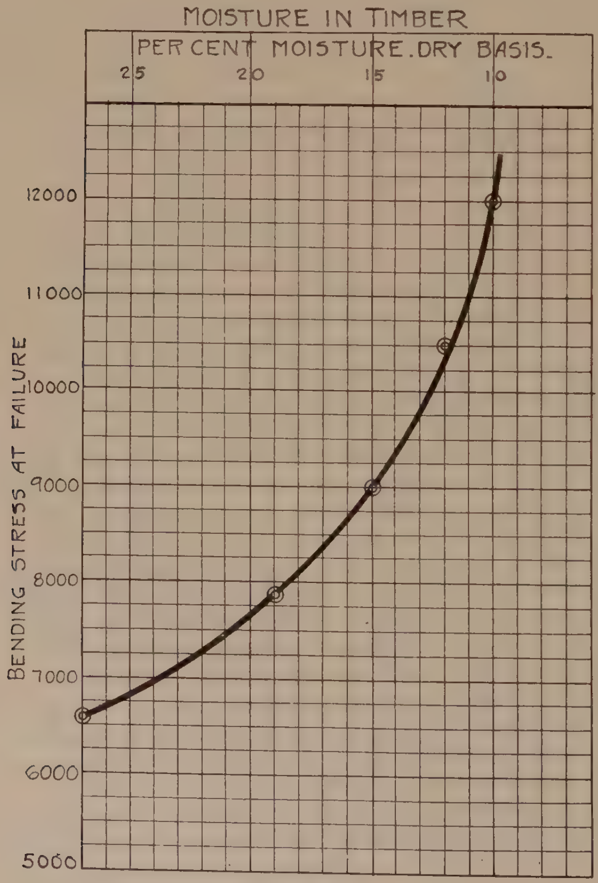


FIG. 2

The average mill buildings for textile and similar uses now have high percentages of moisture in the air created by artificial humidification. In such a case an air-dried timber, after being placed in the mill, does not gain in strength, and at least retains its percentage of moisture (say, 25 per cent). Due to the favorable conditions of heat and moisture for rot fungus growth, such a timber is soon reduced in strength and becomes a necessary subject for inspection. This condition is reached in from ten to twenty years for untreated timber.

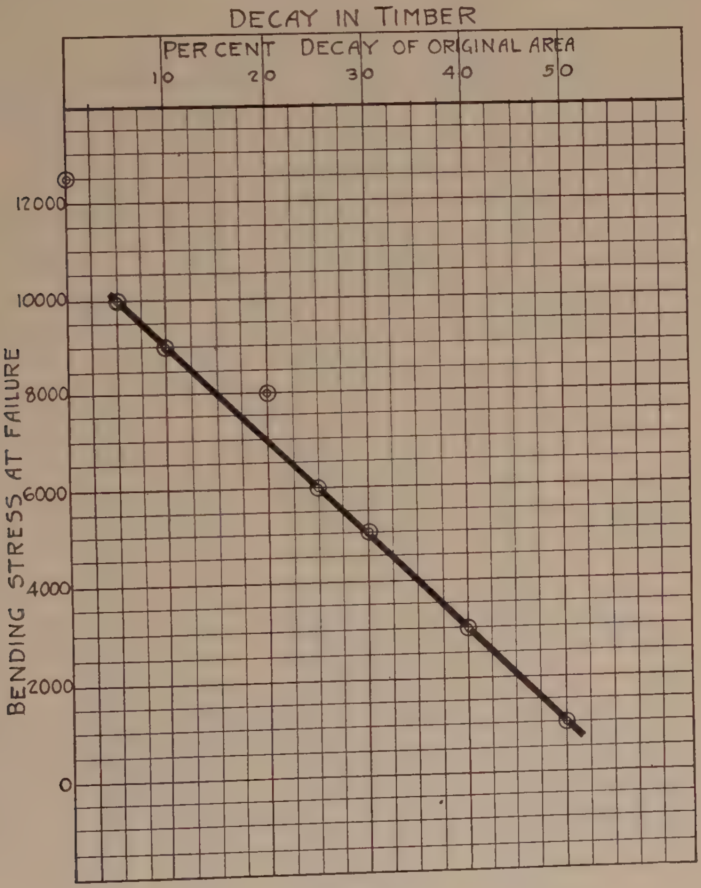


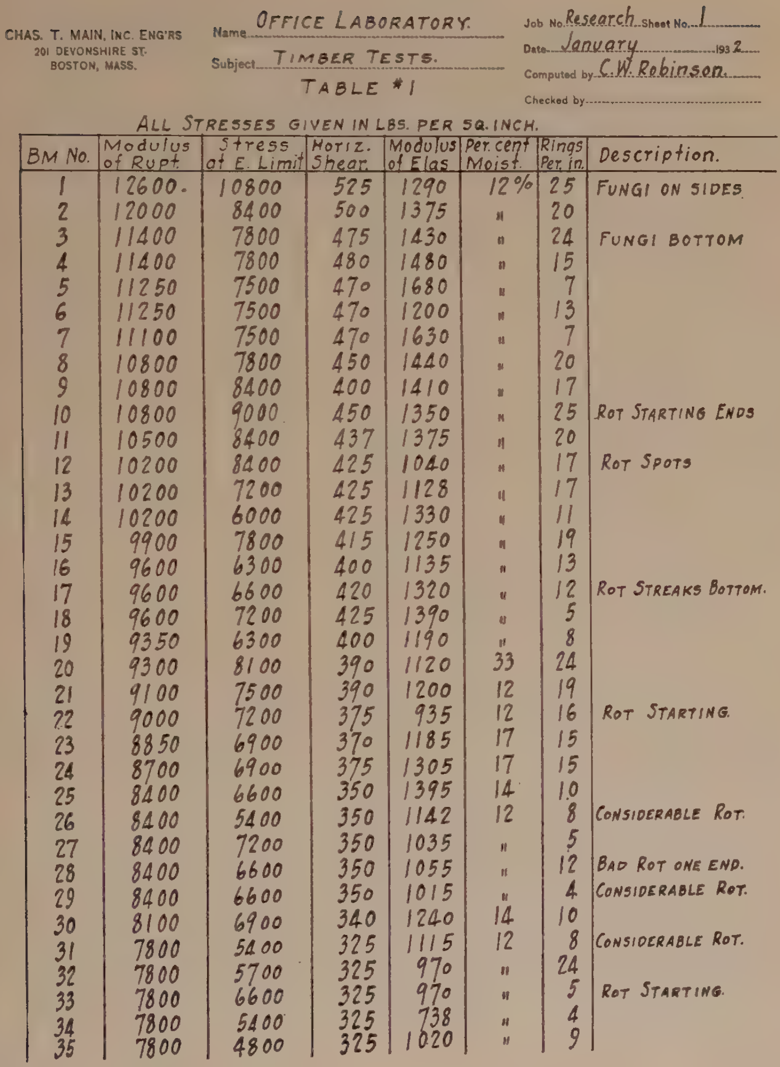
Fig. 3

The safe strength of doubtful timbers, when rot has progressed to a serious point, is determined by obtaining samples of the wood by borings; making a determination of the moisture content; and fixing the progress and location of rot fungus in connection with other defects.

These having been found, comparison may be made with results of bending tests, and the strength may be arrived at quite closely. It may not be accurate, but at least it gives a more exact basis from which to work.

The tests of timber in this series consisted of breaking, by bending, 71 sticks in various stages and conditions of decay. The results are shown in Table 1. This table shows the number marking given the sticks: modulus of rupture; fiber stress at elastic limit; horizontal shear; modulus of elasticity; moisture content; and rings per inch. Accompanying this table are notes describing the failure of each piece, the percentage of rot, its location, etc.

Fig. 3 shows a curve made from results stated in Table 1, and, in general, indicates the bending strength of a timber having only 5 per cent of rot as being about 10,000 pounds per square inch. This was a good timber, and the rot weakened the stick but little. The curve also shows a stick having 50 per cent rot unfavorably placed as having a strength of only 1,000 pounds per square inch.



CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

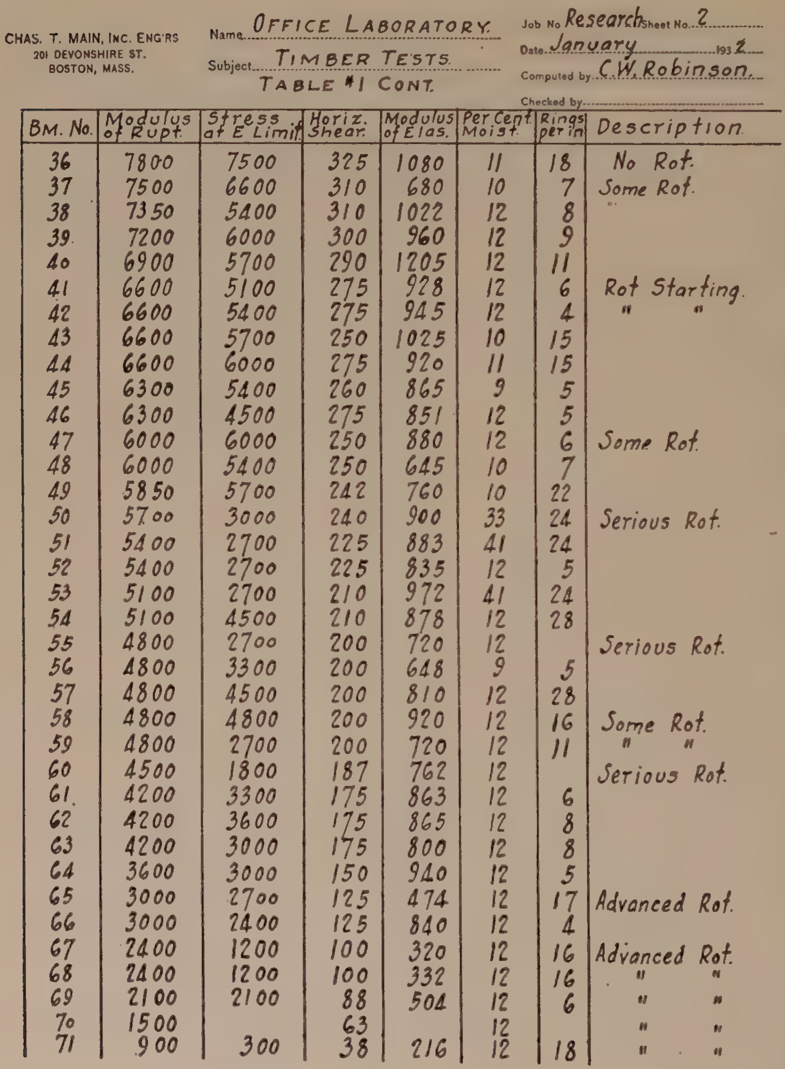
Name OFFICE LABORATORY.
Subject TIMBER TESTS.

Job No. Research Sheet No. 1
Date January 1932
Computed by C.W. Robinson.
Checked by

TABLE #1

ALL STRESSES GIVEN IN LBS. PER SQ. INCH.

BM No.	Modulus of Rupt.	Stress at E. Limit	Horiz. Shear.	Modulus of Elas	Per cent Moist.	Rings Per in.	Description.
1	12600.	10800	525	1290	12%	25	FUNGI ON SIDES
2	12000	8400	500	1375	"	20	
3	11400	7800	475	1430	"	24	FUNGI BOTTOM
4	11400	7800	480	1480	"	15	
5	11250	7500	470	1680	"	7	
6	11250	7500	470	1200	"	13	
7	11100	7500	470	1630	"	7	
8	10800	7800	450	1440	"	20	
9	10800	8400	400	1410	"	17	
10	10800	9000	450	1350	"	25	Rot Starting Ends
11	10500	8400	437	1375	"	20	
12	10200	8400	425	1040	"	17	Rot Spots
13	10200	7200	425	1128	"	17	
14	10200	6000	425	1330	"	11	
15	9900	7800	415	1250	"	19	
16	9600	6300	400	1135	"	13	
17	9600	6600	420	1320	"	12	Rot Streaks Bottom.
18	9600	7200	425	1390	"	5	
19	9350	6300	400	1190	"	8	
20	9300	8100	390	1120	33	24	
21	9100	7500	390	1200	12	19	
22	9000	7200	375	935	12	16	Rot Starting.
23	8850	6900	370	1185	17	15	
24	8700	6900	375	1305	17	15	
25	8400	6600	350	1395	14	10	
26	8400	5400	350	1142	12	8	Considerable Rot.
27	8400	7200	350	1035	"	5	
28	8400	6600	350	1055	"	12	Bad Rot one end.
29	8400	6600	350	1015	"	4	Considerable Rot.
30	8100	6900	340	1240	14	10	
31	7800	5400	325	1115	12	8	Considerable Rot.
32	7800	5700	325	970	"	24	
33	7800	6600	325	970	"	5	Rot Starting.
34	7800	5400	325	738	"	4	
35	7800	4800	325	1020	"	9	



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Name OFFICE LABORATORY.
Subject TIMBER TESTS.
TABLE #1 CONT.

Job No Research Sheet No. 2
Date January 1932
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BM. No.	Modulus of Rupt.	Stress at E Limit.	Horiz. Shear.	Modulus of Elas.	Per Cent Moist.	Rings per in.	Description.
36	7800	7500	325	1080	11	18	No Rot.
37	7500	6600	310	680	10	7	Some Rot.
38	7350	5400	310	1022	12	8	
39	7200	6000	300	960	12	9	
40	6900	5700	290	1205	12	11	
41	6600	5100	275	928	12	6	Rot Starting.
42	6600	5400	275	945	12	4	" "
43	6600	5700	250	1025	10	15	
44	6600	6000	275	920	11	15	
45	6300	5400	260	865	9	5	
46	6300	4500	275	851	12	5	
47	6000	6000	250	880	12	6	Some Rot.
48	6000	5400	250	645	10	7	
49	5850	5700	242	760	10	22	
50	5700	3000	240	900	33	24	Serious Rot.
51	5400	2700	225	883	41	24	
52	5400	2700	225	835	12	5	
53	5100	2700	210	972	41	24	
54	5100	4500	210	878	12	28	
55	4800	2700	200	720	12		Serious Rot.
56	4800	3300	200	648	9	5	
57	4800	4500	200	810	12	28	
58	4800	4800	200	920	12	16	Some Rot.
59	4800	2700	200	720	12	11	" "
60	4500	1800	187	762	12		Serious Rot.
61	4200	3300	175	863	12	6	
62	4200	3600	175	865	12	8	
63	4200	3000	175	800	12	8	
64	3600	3000	150	940	12	5	
65	3000	2700	125	474	12	17	Advanced Rot.
66	3000	2400	125	840	12	4	
67	2400	1200	100	320	12	16	Advanced Rot.
68	2400	1200	100	332	12	16	" "
69	2100	2100	88	504	12	6	" "
70	1500		63		12		" "
71	900	300	38	216	12	18	" "

OFFICE LABORATORY — NOTES ON TABLE 1

Stresses given in pounds per square inch.
Moduli of elasticity given in thousands of pounds.
Hygroscopic moisture given in per cent of dry weight.
Rings given per radial inch.
Detailed description and character of breaks given below.
All beams of southern hard pine 2 by 3 inches tested with 3-inch dimension vertical on a 36-inch span.

1. — Failed in tension followed by compression. No knots. Sappy. Rings diagonal. Wood firm. *Fungi and sap stain* on one side. Otherwise sound.
2. — Failed in splintering tension starting at small knot. Summer wood light.
3. — Tension failure. All sap. *Sap stain. Fungi bottom face.* No knots. Rings diagonal.
4. — Tension failure. Slight spiral grain. Heavy dark resinous summer wood. No knots.
5. — Tension. No defects. Straight grained. Summer wood dark, distinct and resinous. Very heavy stick.
6. — Simple tension.
7. — Same as No. 5.
8. — Simple tension. Heavy dark resinous summer wood.
9. — Simple tension. Light summer wood.
10. — Tension failure followed by compression. Sappy. Sap stain. Rings diagonal. *Rot starting.*
11. — Tension failure. Light summer wood. No knots.
12. — Heavy stick. *Some spots of rot.* Straight grained. Tension followed by compression failure.
13. — Tension failure. Light summer wood. Worm holes.
14. — Simple tension followed by compression failure. No defects. Summer wood distinct. Rings vertical. Straight grain.
15. — Tension break.
16. — Simple tension followed by splintering tension and compression failure. No defects. Summer wood light.
17. — Tension break. No knots. *Rot streak bottom* face. Sap stain.
18. — Tension. Sound, fairly straight grain. No knots. Very light stick.
19. — Tension failure.
20. — Cross grain tension failure.
21. — Tension failure.
22. — Tension failure. Nearly all sap wood. *Rot incipient.* Sap stain.
23. — Cross grain tension failure. Spiral grain. Otherwise a perfect stick.
24. — Cross grain tension failure. Spiral grain.
25. — Tension failure. No knots. Mostly sap.
26. — Tension failure. Heavy, all heart stick. *Lower corner and one end decayed.* Failed at decayed portion.
27. — Tension failure. No knots. Very light stick. Spring wood soft.
28. — Horizontal shear failure. *Rot hole in end and streak along side.* Nearly all sap wood. No knots.
29. — Tension failure. No knots. Sappy. Sap stain. Coarse grain. *Rot somewhat advanced.*
30. — Tension failure starting at ½-inch knot near center of span.
31. — Tension failure. All heart. Heavy stick. *One end rotted.* Balance of stick sound.
32. — Tension failure. All sap wood. Sap stain. *Rot started.* No knots.
33. — Tension failure. Cross grain. All heart work.
34. — Failure in tension followed by failure in compression. Stick coarse. Spring wood soft. One-half sap wood. Wood firm but dark colored.
35. — Tension. Light summer wood.
36. — Tension failure. *Incipient rot.*
37. — Cross grain tension failure. *Some rot.*
38. — Tension failure.
39. — Tension failure. Light summer wood.
40. — Tension failure. Cross grain. Two knots. Failed at knot.
41. — Tension failure. No knots. Sap stain. *Rot beginning.*
42. — Tension failure. Stick brashy and coarse. Spring wood soft. Sap stain. Rot beginning.
43. — Tension failure at knot.

44. — Cross grain tension failure. Summer wood dark.
45. — Cross grain tension followed by complete failure at third point above neutral axis. Large knots at ends.
46. — Tension failure. No knots. Mostly sap. Sap stain.
47. — Tension failure. *Sap stain and rot.* No knots.
48. — Tension failure beginning at knot 6 inches from center line.
49. — Cross grain tension failure.
50. — Cross grain tension failure. Regular grain straight. Secondary grain spiral. Sap stain; *¼-inch rot on bottom face.*
51. — Tension failure. Sap wood spongy and soggy like blotting paper.
52. — Tension failure. Mostly sap wood.
53. — Same as No. 51.
54. — Broke in two parts suddenly. Cross grained. Sound. 50 per cent heart wood.
55. — Tension failure. *Lower face rotted average ⅜ inch deep.* When load was removed stick returned to horizontal and point of failure could not be seen.
56. — Cross grain failure. Instantaneous failure. Large knots at ends.
57. — Cross grain failure. Sudden.
58. — Tension failure. Nearly all sap wood. Sap stain *and some rot.*
59. — Cross grain tension failure. *Some rot.*
60. — Failed in tension and compression at third point *where rot spots were visible on surface.*
61. — Tension failure at third point at knot on neutral axis.
62. — Tension failure. Sound stick, light but all heart.
63. — Failed at knot at center of span.
64. — Failed at knot above neutral axis, 6 inches to right of center of span. Cross grained caused by knot.
65. — *Advanced rot failure* in tension.
66. — Failed at knot across bottom face.
67. — Total failure because of *advanced rot.*
68. — Horizontal shear failure. *Rot advanced.*
69. — *Advanced rot.* Failure in tension.
70. — *Advanced rot.* Failure in tension.
71. — *Advanced rot.* Failure in tension.

Location of defects, including rot, must always be taken into account when fixing values for other structures.

Fig. 4 shows, in diagram, the design of the home-made machine for breaking the sticks. These tests are to be followed by another series in which an even greater number of sticks will be tested by bending and which will fail from advanced age and rot.

There are still a great many mill buildings in New England whose floors are made of wood, and many of these, sooner or later, must either be used at reduced loadings or economically reinforced for the remainder of their useful life.

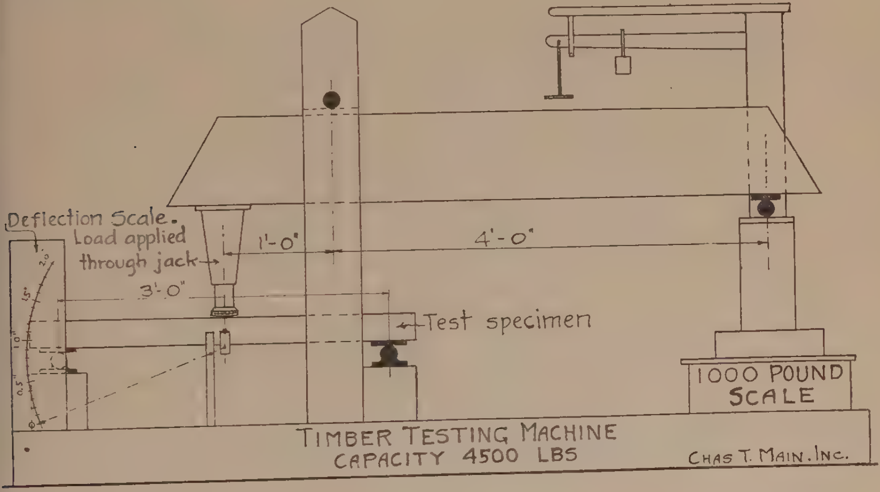


FIG. 4

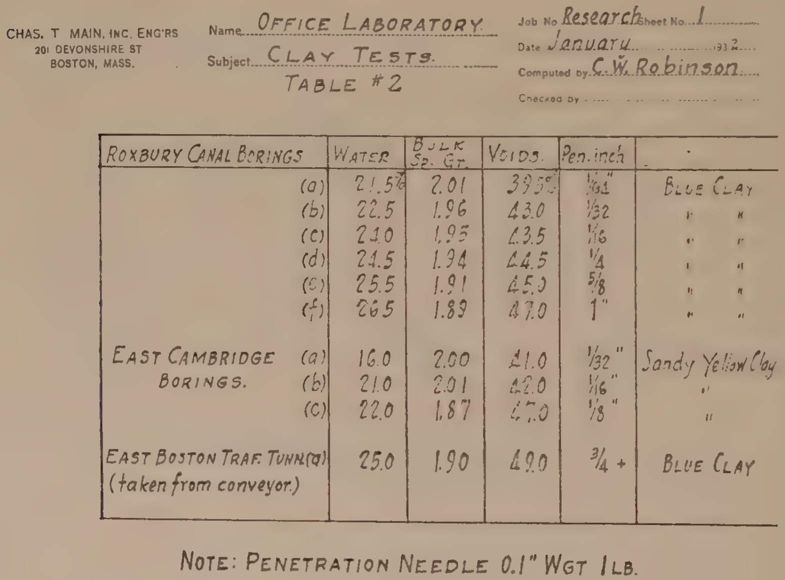
Just outside of this problem the fact may be stated that wood is now seldom put into mills, having high humidity, without being treated to prevent growth of rot fungus, and when so treated its useful life is lengthened from three to four times over the life of untreated wood.

Treated timber will be found to have a useful life about equal to the life of most of the buildings in which it is used, owing to obsolescence being effective in putting most buildings out of the use for which they were designed in much shorter periods of time than used to be the case.

The second subject of research which I will present is "The Classification of Clay for Density." The object of making this series of tests was to find, if possible, some mechanical way of classifying clay as definitely "hard," "medium" or "soft."

All of us, probably, have had the experience of being told, by men who make our borings, that a clay was definitely one classification, and discovering afterwards that the foreman of the gang making the borings had misjudged the soil samples. The methods of arriving at their decisions differ with almost every man in each different company. There has been no one standard for what is "hard" or "medium" or "soft" clay, and where the line is drawn between them.

Wishing to see if this could not be cleared up, I had a series of tests made on different clays to see if the penetration of a needle under a certain weight would not give us some conclusive data. Hundreds of tests have been made on clays taken from the ground in various ways, some representing distinctly disturbed samples, while others represent practically undisturbed clay. A few examples of blue clay are described in Table 2 and penetrations stated.



CHAS. T MAIN, INC. ENG'RS
201 DEVONSHIRE ST
BOSTON, MASS.

Name OFFICE LABORATORY
Subject CLAY TESTS.
TABLE #2

Job No Research Sheet No. 1
Date January 1932
Computed by C. W. Robinson
Checked by

ROXBURY CANAL BORINGS	WATER	BULK SP. GR.	VOIDS.	Pen. inch	
(a)	21.5%	2.01	39.5%	1/64"	BLUE CLAY
(b)	22.5	1.96	43.0	1/32	" "
(c)	23.0	1.95	43.5	1/16	" "
(d)	24.5	1.94	44.5	1/4	" "
(e)	25.5	1.91	45.0	5/8	" "
(f)	26.5	1.89	47.0	1"	" "
EAST CAMBRIDGE BORINGS. (a)	16.0	2.00	41.0	1/32"	Sandy Yellow Clay
(b)	21.0	2.01	42.0	1/16"	"
(c)	22.0	1.87	47.0	1/8"	"
EAST BOSTON TRAF. TUNN. (a) (taken from conveyor.)	25.0	1.90	49.0	3/4 +	BLUE CLAY

NOTE: PENETRATION NEEDLE 0.1" WGT 1 LB.

In this table will be found: water content in percentages; percentage of voids; bulk specific gravity and penetration of a needle 0.1 inch in diameter, having a weight of 16 ounces upon it. Knowing that the physical characteristics control the penetrations, it is interesting to note the penetrations of the needle as compared with the water content, bulk specific gravity, and voids.

Fig. 5 shows a plot in which the penetration curves of water per-entage, voids and bulk specific gravity are given. The needle pene-ration shows such a logical and consistent result that we are led to elieve that some such method as this is feasible in determining the ensity of clay.

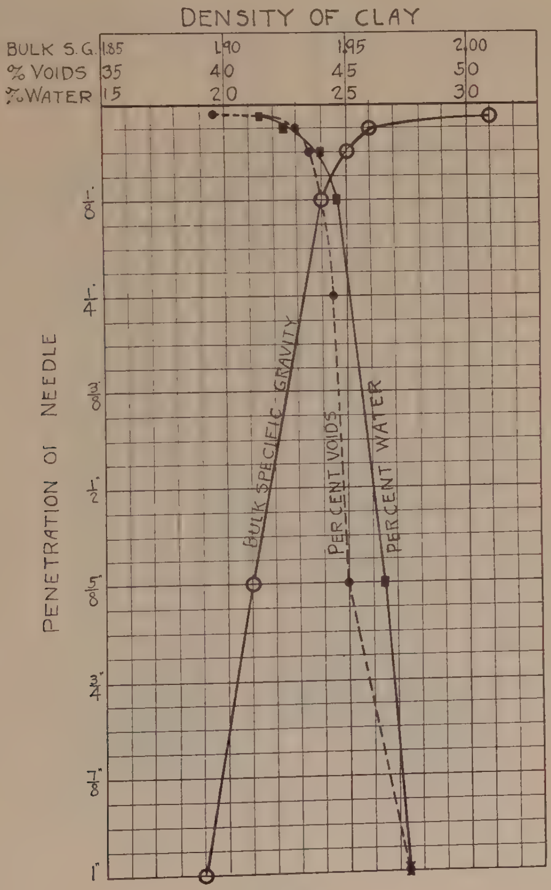


Fig. 5

Fig. 6 is a reproduction of a photograph showing the frame, needle and weight used in making the experiments. The sample to be tested is placed under the needle and the needle moved down until it just touches the sample. The needle is then released by the thumbscrew and penetration results.

It is obvious that if the limits permitted for "medium" clay penetrations are fixed, the "hard" and "soft" clay limits are also fixed. At present it is suggested that when penetrations between one-eighth of an inch and three-fourths of an inch are obtained such clay be classified as "medium."

The report of research work on two subjects of common interest to many of us have just been described for the obvious purpose of encouraging such work. Subjects of greater worth and moment should also be undertaken.

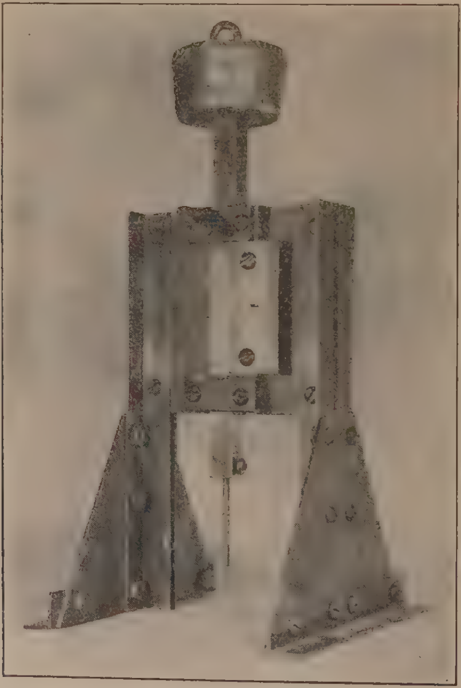


FIG. 6

Table 3 states the facts regarding a typical Boston Blue Clay which was recently given a mechanical analysis by two different methods, *i.e.*, hydrometer and sedimentation. The results were very close and should interest many who are studying clay.

There are not many such analyses of typical Boston Blue Clay, so far as I know. If others are known it would be well to publish them, or, if published, to collect them.

Referring to the subject of this paper, *i.e.*, "Society Research," I will state again that it is probably wise and proper for all engineering schools to teach the fundamentals of science and engineering thoroughly. The young engineer should, however, realize that a liberal education cannot be obtained in any school, but that he must never cease to build for himself. One of the best aids in liberalizing his education after leaving school is to enter upon some kind of original research work in which his originality and ability to think are developed.

TABLE #3

MECHANICAL ANALYSIS	(HYDROMETER ANALYSIS.) ROXBURY CANAL.	(SEDIMENTATION ANALYSIS.) EAST CAMBRIDGE
FINE SAND.	4.%	6.0%
SILT.	36. "	31.0 "
ROCK FLOUR.	30. "	33.0 "
CLAY (TRUE)	30. "	30.0 "

The Society, having published a list of subjects suitable for research, could logically follow this up by asking the committee that made this schedule to sponsor the further work of encouraging our younger members to take on some such work under the committee's guidance. The committee, made up of practicing engineers and professors working in technical schools, could furnish almost ideal assistance in shaping such work toward getting the maximum value out of the investigation for the member and the Society. It seems to me that this work, if undertaken by the Society, would meet the demands of the times and enhance the Society's value to its members and the community.

THE STRUCTURE OF CLAY AND ITS IMPORTANCE IN FOUNDATION ENGINEERING

By Arthur Casagrande *

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers, January 13, 1932)

Introduction

During the past two years a considerable amount of research on the physical properties of clay in the undisturbed state and in the remolded state has been carried out at the Massachusetts Institute of Technology. The results so far obtained seem so important that they should be made available to the engineering profession.

While the conclusions which one is forced to draw from these researches have startled me, and no doubt will surprise many in the audience, I should like to emphasize that there is nothing new in the underlying principles and the testing methods on which these investigations are based. They were developed by Professor Terzaghi during the past ten years and are available in his publications. Much of the material I am about to present to you has been said before without receiving the attention it deserves. This may be due to the fact that modern soil mechanics requires a thorough knowledge of certain phenomena of physics and of advanced calculus for the solution of the problems. I feel that if I start my lecture with a discussion of the capillary pressure in clay due to the surface tension of water, some among the audience would immediately begin to wish they had not come. But I shall try not to use any of these mysterious terms nor any complicated mathematics.

Before I proceed to the main part of this lecture, I am going to discuss in a general way the meaning of the terms stress, deformation and rupture as applied to soil. While there is no doubt what we mean by these terms when speaking of structures built of steel, concrete or other common construction materials, there is some confusion in their application in foundation engineering.

In order to compute the safe bearing load of a beam we must know the dimensions of the beam and the strength of the material of which the beam is made. From these data we compute the load at which the

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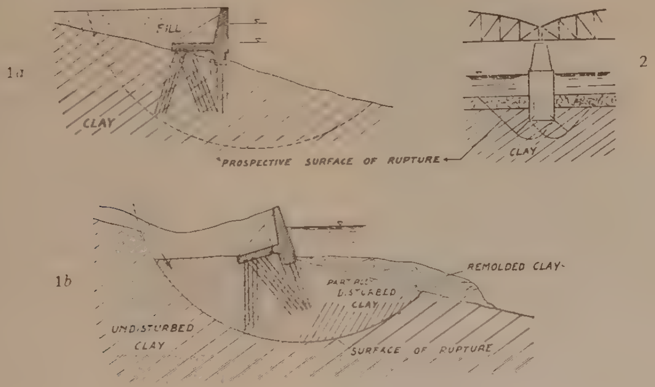
* United States Bureau of Public Roads, stationed at the Massachusetts Institute of Technology.

beam would fail, and after deciding on a factor of safety we can determine the maximum safe load, which we may call the safe bearing load of this beam.

Let us suppose that the beam is very flexible and that, in addition to its load, it carries an accurate instrument which will get out of adjustment if the deflection of the beam exceeds a certain small value. In this case it will be necessary to compute that load which causes the admissible deflection. I shall call this load the allowable load. This allowable load may be very much smaller than the safe bearing load of the beam. If one has to design such beams very often, one should know in advance that it is not necessary to compute the safe bearing load, but that one must determine the allowable load from the allowable deformation; and, in order to compute the deformation, one must know the stress-strain relationship — for example, the modulus of elasticity — of the material rather than its beam strength.



FIGS. 1*a*, 1*b*, AND 2

In structural engineering the number of cases in which the load is limited by deformation is small on account of the great rigidity of structural materials. For soils the opposite is generally the case.

Soil is so much more compressible, and its deformations are so much larger than those of our common construction materials, that in the great majority of foundation problems the loading of the soil must be limited to a value which is much below the safe bearing load of the soil. Those cases where danger of actual rupture of the soil exists are found in engineering structures, such as dams, dikes, quay walls (Fig. 1), in railway and highway engineering, in cuts and fills, retaining walls, masts under wind pressure, etc. In other words, structures with unsymmetrical load distribution on the soil are apt to fail by rupture, and their analysis must be based on the shearing strength of the soil. (1), (2), (3) and (4)*

Among those structures which represent a symmetrical vertical loading, and yet are apt to fail due to rupture of the soil, are bridge piers which have to carry large loads on a small area (Fig. 2). (5) and(6)

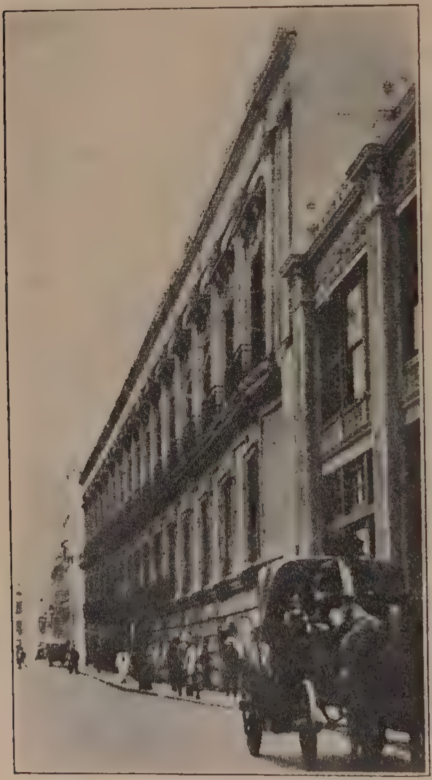
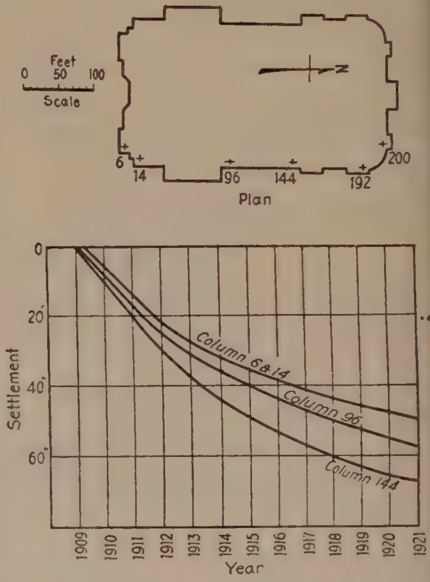


FIG. 3



CURVES OF SETTLEMENT OF VARIOUS COLUMNS, NATIONAL THEATER, MEXICO CITY

THE CURVES SHOW A DIMINISHING RATE OF SETTLEMENT. THE BUILDING HAS A CONCRETE MAT FOUNDATION. THE GREAT SETTLEMENTS ARE DUE TO OVERLOADING OF THE SOIL, WHICH CONSISTS OF VERY FINE, SATURATED, VOLCANIC SILT MORE THAN 600 FEET DEEP, AN OLD LAKE BOTTOM.

FIG. 4

However, in the field of building construction the possibility of actual rupture of the soil is very remote.† I do not know of a single instance where rupture of a homogeneous soil has taken place under a building.

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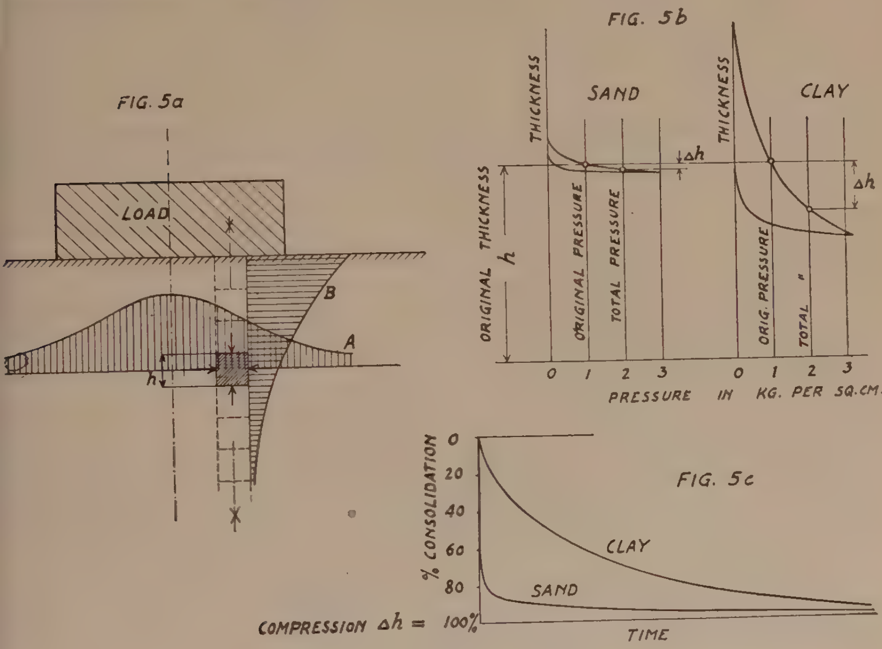
* Numbers in parentheses refer to bibliography.

† Some failures of dams and earth subsidences which are due to the effects of ground water movement are, strictly speaking, not failures of the soil due to overloading. Professor Terzaghi terms such occurrences "underground erosion" and gives an excellent description in (7) and (8).

Even if the settlements amount to many feet, as is often the case in Mexico City, in Constantinople and some other cities, we have no right to speak of failure or rupture of the soil. In Fig. 3 * is shown the old School of Mines in Mexico City, which has settled so much that there is a difference in elevation between the center of the building and its ends of about four feet. The settlement is, nevertheless, due to deformation and compression of the soil. In order to cause actual rupture the loads would have to be many times greater.



Figs. 5*a*, 5*b* and 5*c*

I should like to emphasize that the settlement of a building founded on clay, or certain other fine grained soils, progresses with time at a gradually decreasing rate. Very often we are not interested in the total settlement, which may take several hundred years for its full development, but we are more interested to know the rate of settlement during the first years after construction. In order to illustrate this, let us consider the progress of subsidence of the National Theater in Mexico City, shown in Fig. 4.* The maximum rate of settlement was one foot per year. The settlement during the first ten years amounted to about five feet. From the shape of the time-settlement curve, it is possible to estimate that the total settlement will be about twelve feet, and that this value will be practically reached within twenty or thirty years. Let us now imagine another type of soil which will give the same total settlement, but at such slow rate that the settlement within the first five years will be five inches instead of five feet. Obviously, the second case is much safer from the standpoint of building construction.

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* Figs. 3 and 4 are reproduced, with permission of Mr. Lazarus White, from the new book on "Underpinning," by Prentis and White. (22)

In order to understand the cause of settlements, let us consider one small cube of soil somewhere beneath a loaded area (Fig. 5*a*). The stress due to the load will be distributed on the level of this cube, as shown by curve *A*. In addition to the vertical pressure, the load will produce a horizontal pressure, as indicated by curve *B*. As a result of these stresses, the cube will first undergo a lateral deformation without volume change;* and then in the course of time a gradual volume change will take place, due to squeezing out of water. For the first effect we shall use the term *deformation*, and for the second effect the term *compression* or *consolidation*.

By summation of the deformations and compressions of all small cubes in the same vertical line (*X–X*) we arrive at the settlement of this point of the building.

For those soils which may be considered suitable foundation materials, and for loaded areas of a size normally encountered in building construction, the lateral deformation represents only a small part of the total settlement. For most cases it will be sufficient to determine the compressibility of the soil in order to know how much a building is going to settle.

The relationship between pressure and volume of typical soils is shown in Fig. 5*b*. The compressibility of clay is many times larger than that of sand. In addition, there is a fundamental difference as to the rate of settlement. Sand undergoes most of its deformation immediately after it is loaded. Clay requires a long time for this process. Typical time-settlement curves for sand and clay are shown in Fig. 5*c*. The differences in compressibility and in the time effect are even greater than shown in the sketches, as it was necessary to exaggerate the sand curves in order to show the shape at all.

I should like at this point to summarize the foregoing introduction: The same principles which govern the design of our superstructure apply also to the ground on which such a structure is built. Just as we must know the strength and stress-strain relationship of our construction materials, as well as the stress distribution in the individual members of the superstructure, it is also necessary to know the same factors for the soil beneath if we want to determine the safety against rupture and the settlement of the structure due to compression and deformation of the soil. The only difference is that in the superstructure the deformations are so small that we try to utilize for economic reasons the strength of the material within safe limits, while for soil the deformation and compression are so large that, especially in building construction, we cannot utilize its safe bearing load but must be satisfied with a smaller load in order not to exceed a reasonable settlement. While in designing a structure we may often get along by knowing only the strength of the construction material, we are, in the case of soil, obliged to know, first of all, its stress-strain relationships.

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* The volume change of the constituents of clay (mineral grains and water) is negligible under the stresses which we have to consider. We assume that the clay does not contain any noticeable amount of air, which is generally the case in undisturbed clay deposits.

The Compressibility of Clay and its Measurement

We have recognized that settlements of buildings on clay are caused mainly by compression, that is, a volume decrease of the soil due to the superimposed load. If the voids of clay were filled with air, the compression would be almost instantaneous, because the air could escape with very little resistance, simultaneously with the volume decrease of the soil. However, water meets so great a resistance in the very small voids that it takes considerable time until all the excess water is squeezed out. The lower the permeability of the clay and the thicker the compressed clay layer, that is, the longer the path of percolation, the more time will it take for the soil to consolidate under the new load.

If we can succeed in determining the amount of compression a clay will undergo under a load, and the rate at which this compression will proceed, we have also solved the problem of how to determine, at least approximately, the settlement which a building will experience when founded on this clay.

Professor Terzaghi has developed a testing method which allows the determination of the compressibility of clay samples. (9) and (10) A cross section of the apparatus is shown in Fig. 6. The soil sample is pressed between two porous stones which permit free drainage of the water from both faces of the sample. The upper stone forms the base of a piston through which the loads are applied to the sample. The compression of the sample is measured by means of very sensitive extensometers. The loading device forms a separate unit, of which several different types are in use, for different ranges of pressure. They apply a constant load which is increased or decreased by increments (Fig. 7).

When a sample of soft clay is compressed in this so-called consolidation device, water will be squeezed out and the sample will decrease in thickness at a gradually diminishing rate. When movement of the dials stops, the grain structure has reached such a density that it can carry the load without further compression. If, then, another load increment is put on, the soil will compress further until the structure reaches so dense a state that it can carry the additional load. By repeating this process up to the desired maximum pressure, and then decreasing the load by steps to zero, we arrive at a curve (Fig. 5*b*) of logarithmic shape which represents the relationship between pressure and thickness of the sample or void ratio.* The rebound is usually only a fraction of the compression.

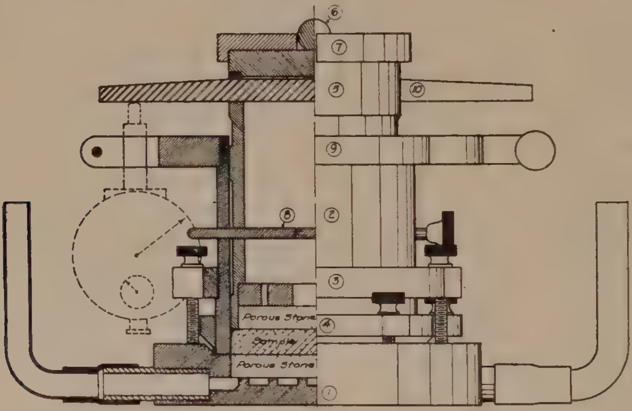

FIG. 6

By making time observations on the extensometers, it is possible to get the relationship between compression and time, as shown in Fig. 5*c*. It is customary to set the total compression under one load increment equal to 100 per cent. From one such time curve for a sample of known thickness h_1, it is very easy to derive the time curve for a sample of another thickness h_2, or for the whole thickness of the clay deposit in nature. All we have to do is to increase the time scale by the factor $\frac{h_2^2}{h_1^2}$. If a sample of one-half inch thickness reaches 90 per cent consolidation in 10 hours, then a soil layer of 100 feet thickness would require 670 years to reach the same degree of consolidation.

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* Void ratio is the volume of voids divided by the volume of the solid matter.

For many years clay samples were tested in a remolded state. It seemed too difficult and costly to remove large undisturbed soil samples from drillholes. Therefore samples were taken in pipes of 1 inch to 2 inches in diameter and 1 foot to 2 feet long. It was realized that this method of taking samples disturbed, at least partially, the natural consistency of the soil without, however, changing the water content. If a sample were compressed in the apparatus in the disturbed state, starting from the original water content W_1, one would get a compression curve A (Fig. 8) which did not go through P_1, the point corresponding to the condition of the soil in nature. The compression curve for the undisturbed soil in the ground is represented by curve B. It was obvious that the sample in the testing apparatus was more compressible than in the undisturbed state in the ground. In order to reproduce the original curve B, the practice developed to add water to the sample and to mix it to a very soft consistency, corresponding about to Atterberg's liquid limit (11). Then, a compression test was started from this state, and this usually would yield a curve C, which was above A but still below the original curve B.

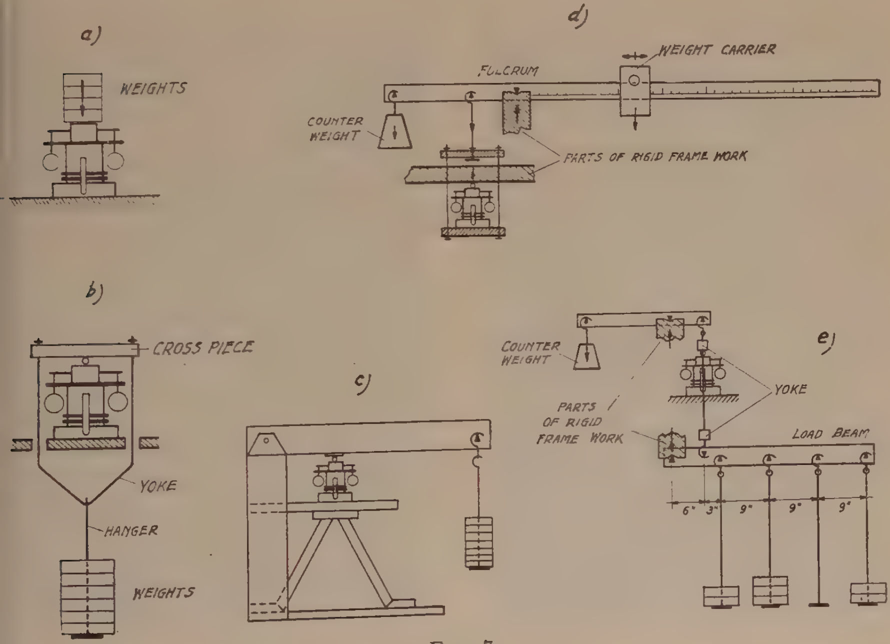

FIG. 7

If it were possible to test an entirely undisturbed sample, it should not compress to any noticeable degree when increasing the pressure from zero to that pressure P_1, under which the sample was consolidated in the ground. A further increase in pressure should cause a compression along the compression curve *B* for natural consolidation.

Two years ago a large undisturbed cube of clay from the Laurentian Valley was received through the courtesy of Prof. C. B. Breed. This clay was well consolidated, having a cube strength ranging between 30 and 70 pounds per square inch. When a piece was remolded it would first break and crumble between the fingers as if it had no plasticity. When squeezed with increasing pressure it would suddenly yield and become plastic and sticky. Most clays in the undisturbed state, which were never exposed before to the influence of outside conditions, display such a behavior, at least to a certain degree.

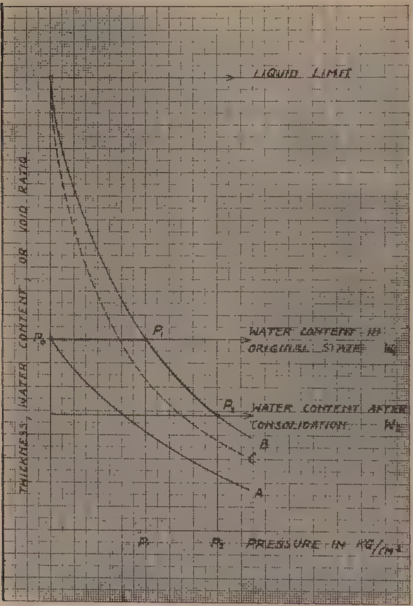

FIG. 8

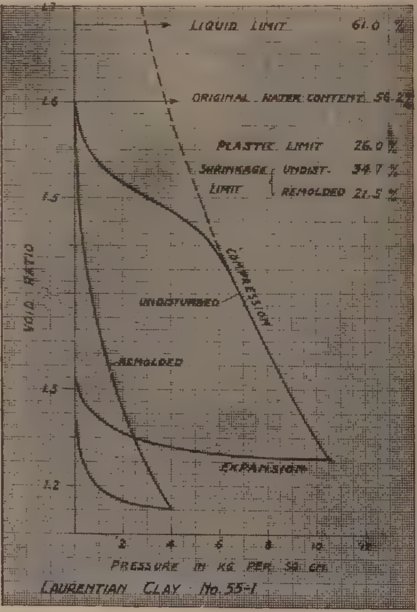

FIG. 9

From this undisturbed clay a disc of the proper size was carefully cut out and submitted to a consolidation test. The material immediately surrounding this disc was thoroughly remolded and placed in a second apparatus for testing. These operations were carried out in such a manner as to prevent any loss of moisture. The results of these compression tests are shown in Fig. 9. The undisturbed compression curve shows, at least approximately, the characteristics which one should expect from the previous discussion. The reason why the first part of the curve, before the original compression curve is reached, is not horizontal will be explained later. By continuing the steep part of the compression curve backward (dotted line) we arrive at the curve along which nature has consolidated this deposit. It appears that this original compression curve must start considerably above the liquid limit, at a water content which would correspond to a practically liquid state if it were reproduced in the laboratory. At a consistency corresponding to the liquid limit the natural deposit can support a load of four tons per square foot without appreciable compression, while in the laboratory it would undergo excessive settlements at a fraction of this load, as we can see from the compression curve in the remolded state. Apparently nature builds up such clay deposits in a peculiar structure which is so carefully arranged that it is many times stronger than the irregularly arranged grains in the remolded state, or that structure produced in the laboratory by consolidation starting from or above the liquid limit.

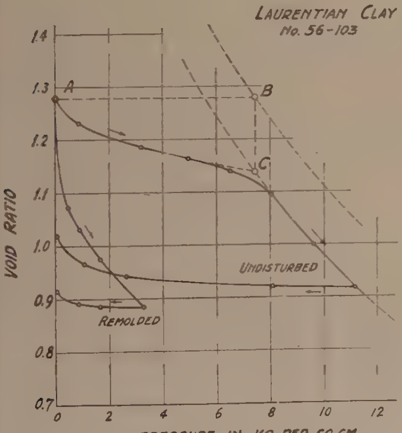

FIG. 10

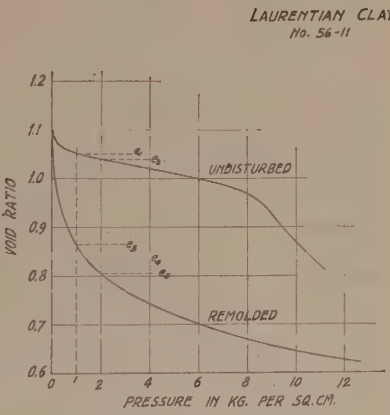

FIG. 11

After these results were obtained it was concluded that the shrinkage limit* in the undisturbed state should be considerably larger than

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* The shrinkage limit is the water content of clay in the state when it has reached its smallest volume during a drying process. (11)

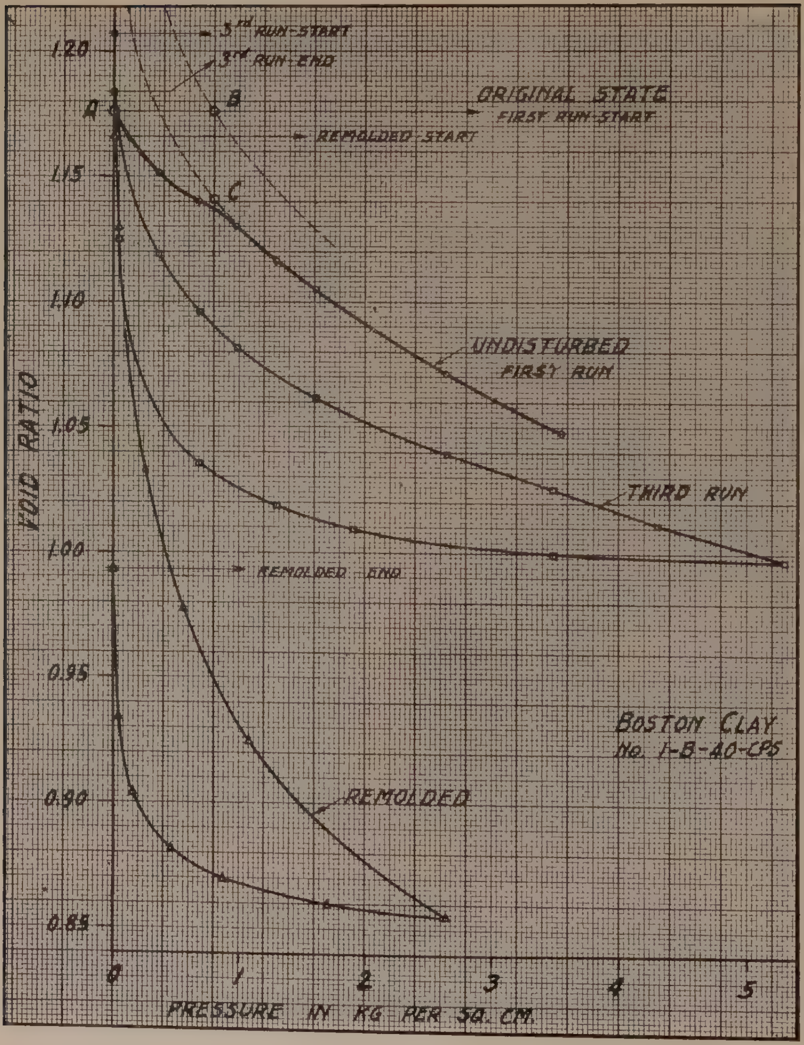
5th RUN-START
3rd RUN-END
B
A
ORIGINAL STATE
FIRST RUN-START
REMOLDED START
C
UNDISTURBED
FIRST RUN
THIRD RUN
REMOLDED END
REMOLDED
BOSTON CLAY
No. 1-B-40-CP5
VOID RATIO
1.20
1.15
1.10
1.05
1.00
0.95
0.90
0.85
0
1
2
3
4
5
PRESSURE IN KG PER SQ. CM.

FIG. 12

e shrinkage limit in the remolded state. The test results confirmed is assumption. The shrinkage limit in the undisturbed state was .7 per cent as against 21.5 per cent in the remolded state. In other ords, the volume of voids of a sample dried from the undisturbed state as 50 per cent larger than for the remolded and dried sample.

Later, Mr. Irving B. Crosby* gave me the opportunity to test a umber of undisturbed clay samples from the same formation but a fferent locality. Typical test results are shown in Figs. 10 and 11. gain we can notice the remarkable difference between the compressility in undisturbed and remolded states. It appears that these samples ere consolidated under pressures .between 7 and 9 kg./cm.^{2}. Mr. rosby has found geological evidence which confirms these figures.†

Under Professor Gilboy's supervision, a number of samples of clay om two different localities in the vicinity of Boston were tested. In ig. 12 are shown the compression and expansion curves of one sample undisturbed and remolded states. For this material the difference in ompressibility between the two states is considerably smaller than for ne clay samples from Canada, represented in Figs. 9, 10 and 11. In ddition, there is only an indication of a break in the compression curve the undisturbed sample corresponding to the pressure at which conolidation has taken place in the ground, while for the Laurentian clays ne break was very distinct. However, even with the Laurentian amples, there was a noticeable compression of the sample up to the onsolidation pressure. There are several reasons for this partial or omplete disappearance of the break in the compression curve of the ndisturbed material. Shortly before the sample is removed from the ottom of the drillhole it will have a tendency to swell by absorbing ater from the surrounding clay, and especially any water which may eep into the drillhole despite all precautions. No matter how carefully sample is removed from the soil and prepared for the test, there will lways be a certain amount of disturbance connected with these operaions. It is practically impossible to cut the sample to a prefect fit in he compression apparatus. Consequently, there will be a non-uniform ressure distribution at the start of the test. While the average pressure ver the whole area of the sample is still below the consolidation presure, locally this pressure is exceeded and causes a premature comression of part of the sample. Most important, however, seems to be he effect of *internal swelling*, which will be explained under the following eading.

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* Consulting Geologist, Boston, Mass.

† This seems to open up a new avenue of approach for geological investigations of sedimentary deosits.

A TENTATIVE EXPLANATION OF THE STRUCTURE OF A NATURAL CLAY DEPOSIT

So far, no explanation has been given for the cause of the difference in the physical properties of clay in undisturbed and remolded states. From the standpoint of application for the purposes of foundation engineering, it would not be absolutely necessary to know the reason. Just as improvements in the properties of steel or cement were often found empirically and utilized in engineering long before the physical significance was understood, we could utilize the knowledge of the compressibility of clay in undisturbed and remolded states in foundation design. But if we can find an explanation which checks with the observations made so far, we shall welcome it. The following hypothesis seems to explain the observed phenomena in a satisfactory way.

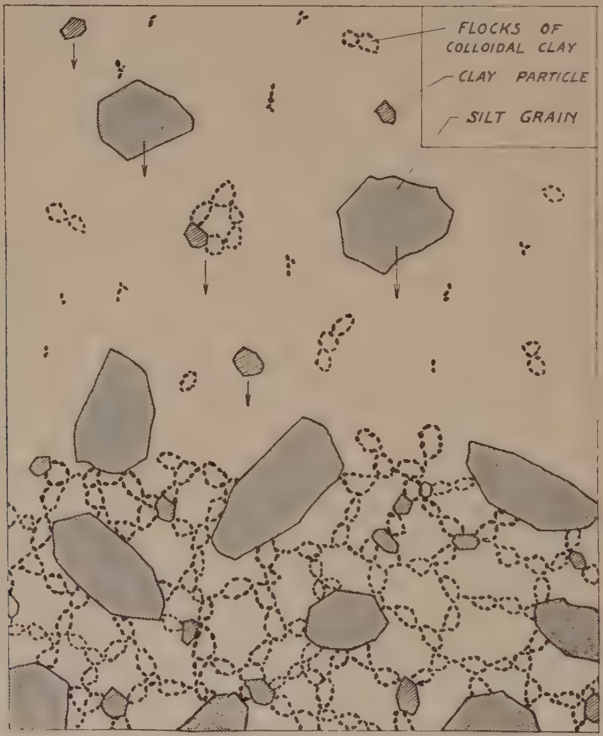


FIG. 13

During the process of sedimentation in fresh water, the silt and arger clay particles settle out individually, while clay particles of coloidal size first form flocks which are large enough to overcome the effect f Brownian movement (5) and then to settle out. In sea water the oagulating effect of the salts is so strong that it causes, also, clay paricles of larger size and even silt and small sand grains to form flocks uring their downward movement.

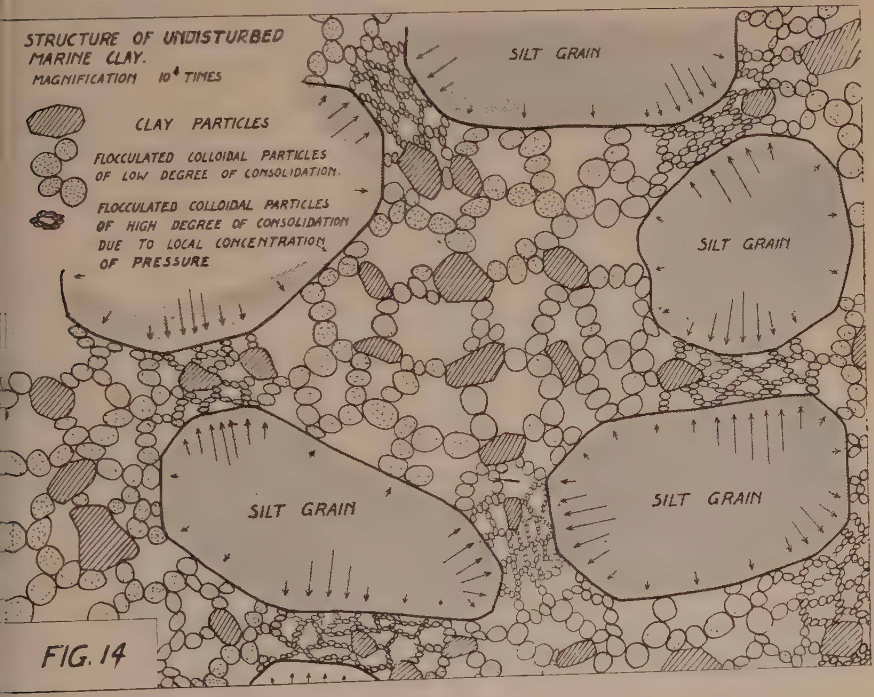

STRUCTURE OF UNDISTURBED MARINE CLAY.
MAGNIFICATION 10^4 TIMES
CLAY PARTICLES
FLOCCULATED COLLOIDAL PARTICLES OF LOW DEGREE OF CONSOLIDATION.
FLOCCULATED COLLOIDAL PARTICLES OF HIGH DEGREE OF CONSOLIDATION DUE TO LOCAL CONCENTRATION OF PRESSURE
SILT GRAIN
SILT GRAIN
SILT GRAIN
SILT GRAIN
FIG. 14

Fig. 14

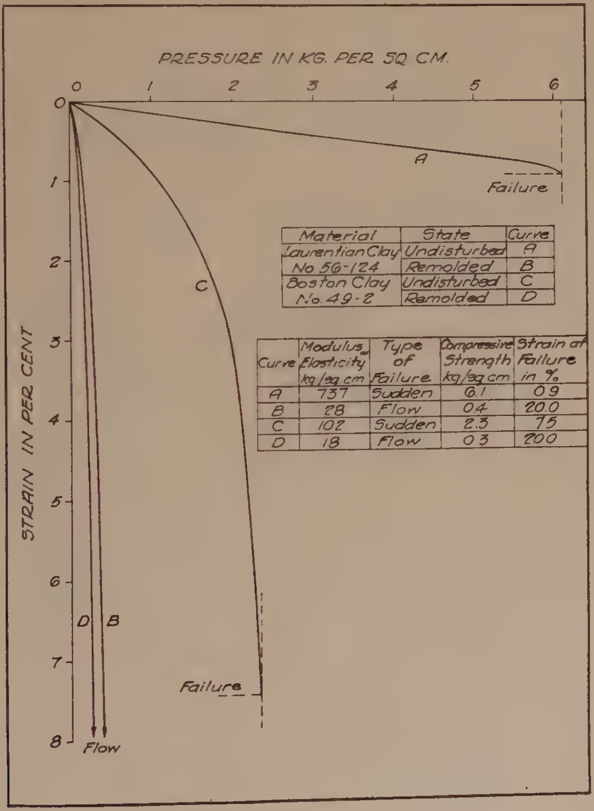
When these flocks and individual grains reach the bottom they form a very loose, honeycomb structure (Fig. 13). They do not settle into a more stable position, but usually remain in that position in which they first touched the bottom. This behavior is due to molecular attraction at the points of contact, which, for grains of this small size, is of the same order of magnitude as the weight of the grains themselves. This process of sedimentation and the formation of the honeycomb structure are shown in Fig. 13, and described in detail by Terzaghi in (5).

Under the steadily growing weight of the overlying sediment consolidation takes place, which means that water is being squeezed out, and the structure becomes more dense. The flocculent structure of the clay in the space where the silt grains are relatively near together should for geometrical reasons undergo a larger compression per unit of volume than the clay contained in the spaces between the grains. A sudden increase in pressure would cause lateral flow and produce the same degree of compression of the clay in the larger voids as in the narrow spaces between the larger grains. However, a very slow increase in pressure allows sufficient time for a higher degree of consolidation to occur in the narrow spaces without lateral flow of the clay. Consequently, the void ratio of the clay in the narrow spaces will decrease more than in the large voids, and the pressure will be carried mainly by the structure of the larger grains, being transferred through the highly consolidated clay in the narrow spaces between these grains. This state is shown in Fig. 14. That part of the clay which fills the space within the individual arches of the structure will carry only a very small fraction of the pressure and therefore will remain in a very soft state, while the consolidated *bond clay* between the grains will carry a unit pressure much higher than the average unit pressure over the whole area.

I should like to emphasize that we have not succeeded, so far, in reproducing this natural structure in the laboratory. It is very unlikely that we shall ever be able to do so, because the building up of such a structure is chiefly dependent on the exceedingly slow process of natural sedimentation and consolidation. If we destroy the structure which nature has taken many centuries to build up, we cannot restore it.

On the basis of the above explanation, the difference in the physical properties between the undisturbed and remolded states is easily understood. Remolding destroys the bond of highly consolidated clay between the silt grains and causes the soft clay filler to surround the larger grains which have formed the structure. As a result, the consistency in the remolded state will be very soft — in extreme cases even above the liquid limit.

As long as the bond of consolidated clay is not broken, the material is fairly elastic and has a considerable strength when loaded in an unconfined state. The strength depends mainly on the pressure under which consolidation has taken place in nature. The strain at failure decreases with increasing consolidation pressure and with increasing ratio between the stresses in the bond clay and the average stress. The more complicated the structure is, the larger the ratio will be. We should expect it to be especially large for clays which were deposited in salt water where thorough coagulation of the suspended clay has taken place before sedimentation; and, indeed, we find that the clays from e Laurentian Valley, which are salt water deposits, are so brittle that ey break at a strain of only one-half to one per cent, a rigidity comarable to that of concrete. Equally interesting is the fact that the ress-strain relationship is practically a straight line, which means that is material is almost perfectly elastic up to the point of rupture (Fig.



Material	State	Curve
Laurentian Clay No 56-124	Undisturbed	A
	Remolded	B
Boston Clay No. 49-2	Undisturbed	C
	Remolded	D

Curve	Modulus of Elasticity kg/sq cm	Type of Failure	Compressive Strength kg/sq cm	Strain at Failure in %
A	737	Sudden	6.1	0.9
B	28	Flow	0.4	20.0
C	102	Sudden	2.3	7.5
D	18	Flow	0.3	20.0

FIG. 15

5). It is really more elastic than either steel or concrete. Steel is elasic only up to the yield point, while the stress-strain diagram for concrete s a curve; that means it is not elastic.

Clays which are consolidated under only one or two tons per square oot, instead of the five to ten tons per square foot of the Laurentian lays, will not exhibit these properties in such a striking degree. But ven samples of Boston clay consolidated at a depth of only forty feet appear fairly hard and elastic if compared with the consistency after remolding (Fig. 15).

The large difference in compressibility of a confined sample between undisturbed and remolded states is easily understood when considering the effect which remolding must have on the natural structure of clay. Before remolding, the load was carried by the structure formed of the larger grains and by the highly consolidated bond clay, while the soft filler carried almost no load. After remolding there is no such structure; only individual grains floating in a mass of almost liquid clay. If the clay were remolded in its original position in the ground, the whole load would have to be carried by the soft clay filler, which would have to adapt itself to the pressure by a considerable decrease in volume; or, in other words, the soft clay must consolidate under the load of the whole overlying soil, while in the undisturbed state it had to carry only a very small percentage of this load.

The difference in compressibility between undisturbed and remolded states varies considerably in different clays. The magnitude of the difference may be considered a measure for the structural effect. It is likely that for a simple honeycomb structure the difference will be smaller than for a structure of that type shown in Fig. 14, consisting of a system of large honeycombs which incorporates the coarser grains, and of a honeycomb structure of second order, consisting of small clay grains and flocks of coagulated colloidal clay. These flocks in themselves represent a honeycomb structure of a still smaller order.

From the standpoint of the sampling and testing operations, it is important to know what happens within the clay when the external load is removed. Just as a spring or a sponge which is compressed will expand again when the load is removed, clay will slowly expand or swell when it is taken from the ground and immersed in water. However, when kept away from water it cannot expand because the surface tension of the water in the microscopic voids is large enough to prevent any expansion. Thus we should expect that the internal stresses in an undisturbed sample, immediately after it is taken from the drillhole, do not change as long as there is no volume increase due to swelling. But even then, if no external water is available which the clay could absorb, following its tendency to swell, there is one possibility of a gradual reduction of the internal stresses by *internal swelling*. The highly consolidated *bond clay* tends to expand with a greater force than the soft clay filler within the structure. In other words, there is a tendency in the structure to deform in such a manner that the total volume of the material remains unchanged, but that the bond clay expands by drawing water from the soft filler. Such a deformation within narrow limits is physically very well possible, and there appears no reason why this internal swelling should not take place.

Now let us consider what will happen when the sample is compressed in the consolidation apparatus. If there were no internal swelling, then there should be no compression of the sample up to the pressure under which it was consolidated in the ground. If, however, internal swelling has taken place, then the bond clay will begin to compress at much lower pressures (Figs. 9 to 12). The reverse process of internal swelling cannot take place because the sample is confined during the test and not free to deform like an unconfined sample. To what extent internal swelling or other causes, like non-uniform pressure distribution at the start of the test, participate in the initial compression is difficult to decide. But we do know that clays, such as the Boston clay, which swell appreciably under water show a much less distinct break in the undisturbed compression curve (Fig. 12) than the Laurentian clays (Figs. 9, 10 and 11), which swell only very little under water. It is, therefore, probable that also internal swelling is more important in the Boston clay than in the Laurentian clays.

If we cannot determine the undisturbed compression curve exactly in accordance with the behavior of the clay in the ground, we like to know, at least, how far our test results may deviate from the correct curve. As an example, we examine the test results on the sample of Boston clay, shown in Fig. 12. The void ratio in the original state is represented by point *A* on the vertical axis. The compression curve in the undisturbed state has a slight break at 0.8 kg./cm.^{2}, which corresponds to the load under which the sample was consolidated in the ground. If the compression in the first part of the curve (from *A* to *C*) were exclusively due to the effect of internal swelling, then the correct or natural compression curve should be about like the dotted line through point *B*. If external swelling and unequal load distribution were the only causes, the natural curve would be represented by the dotted line through *C*, and would continue as part of the experimentally determined curve. The real position lies somewhere between these two dotted curves, and probably nearer to the lower curve which passes through point *C*. The same considerations apply to the tests on Laurentian clays. In Fig. 10 the correct curve lies probably nearer to point *C* than point *B*. The uncertainty in the position of the correct compression curve for the undisturbed state is of little importance when compared with the large difference between the position of the compression curves in the undisturbed and remolded states.

It is interesting to note that repeated loading and unloading of the undisturbed sample causes the compression curve to drop. In Fig. 12 is plotted only the first and the third run of five repetitions in order not to confuse the drawing. A tentative explanation of this phenomenon is that a rapid consolidation causes a breakdown of some of the arches of which the structure is formed. The fact that the second and third compression curves started above the original water content has no special significance. It only means that the undisturbed sample will swell beyond the original water content if water is available.

The amount of compression recovered during the rebound is almost 100 per cent in the undisturbed state, and about one-third or one-half in the remolded state. This is in agreement with our explanation of the natural structure of clay. As long as the structure remains intact, that is, as long as the bond clay is not broken in any point, a clay specimen should be almost perfectly elastic, both in unconfined and confined compression tests.

THE REMOLDING EFFECT OF PILE-DRIVING ON THE STRUCTURE OF CLAY

Before I proceed to the application of these findings to the analysis of the settlements of buildings, we must discuss the extent of disturbance of the structure of clay which is caused by pile-driving. In order to get conclusive data on this point some elaborate investigations are required. It would be necessary to drive single piles and pile groups into undisturbed clay. Then test shafts should be dug along the piles and groups of samples taken at different levels and tested for compressibility. In each level the samples should be taken at different distances from the pile, with at least one sample far enough away to be undisturbed. At present such data are not available, and therefore we must form our opinion from indirect information which I am basing on the following data:

1. Considerations of the character of the movement of clay which is displaced by the pile, and which is pressed upward between the piles (Fig. 16).

2. The increase in driving resistance after the pile rests in the clay for some time, which is due to partial consolidation of the remolded clay surrounding the pile.

3. Observation of the vibrations and the slight lateral flexure at each blow, and of the quaking and the jerky upward movement of the soil between the piles, carried out during the construction of a wooden pile foundation on Boston clay.

As a result, I arrived at the following approximate estimate for piles of common dimensions: the clay immediately surrounding the pile to a distance of ½ diameter of the pile is completely remolded, and to a distance of 1½ diameters it is sufficiently affected to result in a large increase in compressibility of the clay.

It is well known that floating piles in clay and certain silty soils transmit their load to the soil largely through skin friction. I need only to refer to the extensive investigations by Terzaghi (12) and by the Whangpoo Conservancy Board (13). From these it follows that the point resistance in clay never exceeds 20 per cent of the total load at failure. At smaller loads the percentage decreases, and it is probable that for safe loads the whole pressure is transmitted by skin friction.

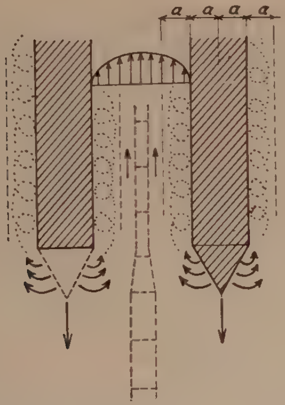


FIG. 16

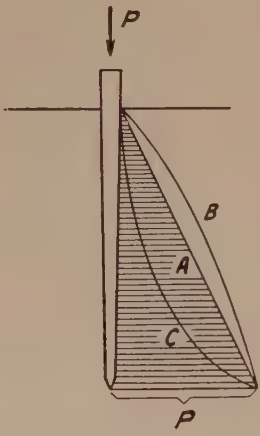


FIG. 17

Although we know definitely that the load on the piles is transmitted into the clay chiefly by skin friction, we have no information on the distribution of the skin friction along the piles. This is an important point and will require elaborate experimental investigations on a large scale before any reliable assumptions can be made. This problem is unusually complicated, due to the fact that the intensity and distribution of skin friction changes with the progress of consolidation of the clay between the piles. Unless the piles themselves facilitate the drainage, the consolidation will proceed from the surface downward, so that the shearing resistance of the clay, and therefore the skin friction along the upper portion of the piles, will increase before a similar increase can take place along the lower section of the piles. On the other hand, it is likely that in the course of time the relative movement between the piles and the soil will mobilize a greater intensity of skin friction along the lower section of the piles, where this relative movement is largest.

One must also consider the shape of the piles. Conical piles transmit more of their load into the upper portion of the soil layer than cylindrical piles.

In the absence of more definite information we are forced to base our assumptions on purely theoretical considerations which have led me to believe that that portion of the total load which the soil at any given elevation has to carry can be represented, in the beginning, by a straight line, as shown by line *A* in Fig. 17. During the first part of the consolidation process the distribution will gradually change to a convex curve, as shown by curve *B*, and during the later part into a concave curve, as shown by *C* in Fig. 17. For the following computations we shall assume a straight line distribution.

While the gradual distribution of the load which rests on the piles, into the soil, tends to decrease the total amount of settlement, the increase due to partial remolding of the clay outweighs this decrease by far. *The major portion of this increase in settlement* is not due to the load which is resting on the piles, but *is due to the consolidation of the remolded soil under its own weight.* The presence of the piles may retard this part of the compression, but it cannot reduce materially its total amount.

SETTLEMENT OF BUILDINGS FOUNDED ON PILES AND WITHOUT PILES — A COMPARISON

The Boston clay, represented by Fig. 12, is consolidated under its own weight of 0.8 kg./cm.^{2}. An additional load of 1.0 kg./cm.^{2} will cause a compression from a void ratio of 1.14 to 1.10. If the clay were completely remolded it would compress under its own weight from 1.14 to 0.95, and the additional load will cause a further compression to a void ratio of 0.88. The compression in the remolded state would be six and one-half times larger than in the undisturbed state. Due to the linear load distribution along the piles, only one-half of the decrease from 0.95 to 0.88 should be taken into account, which would reduce the increase in compression to five and one-half times. A further reduction is due to the fact that the material between the piles is not completely disturbed. The percentage of disturbed material depends chiefly on the spacing of the piles. Even if we make a liberal allowance for any undisturbed and partially disturbed material, we cannot expect that reduction in compressibility will be more than 50 per cent, but probably much less. A reduction of 50 per cent would mean that the clay between the piles would compress two and one-half or three times as much as without piles.

As far as the settlement of the clay below the piles is concerned, it is also increased by the presence of the piles, due to the fact that the load is not spread over a wider area as would be the case without piles.

In the case of the Laurentian clays, the unfavorable influence of piles is even more striking. Let us consider a clay as represented by Fig. 11, which was consolidated under a pressure of about 8 kg./cm.^{2}, and, by erosion, has lost most of the overburden and is exposed to a pressure of 1 kg./cm.^{2}. If the load is increased by the weight of a building to 2 kg./cm.^{2}, the soil will gradually compress from a void ratio of $e_1=1.050$ to $e_2=1.040$. If the soil were completely remolded, it would have to undergo a compression under its own weight from $e_1=1.050$ to $e_3=0.865$, and, due to the additional load, to $e_5=0.805$. For the remolded state the compression would be twenty-four times larger than for the undisturbed state, or, with proper corrections for load distribution along the piles and partial disturbance, about eleven times.

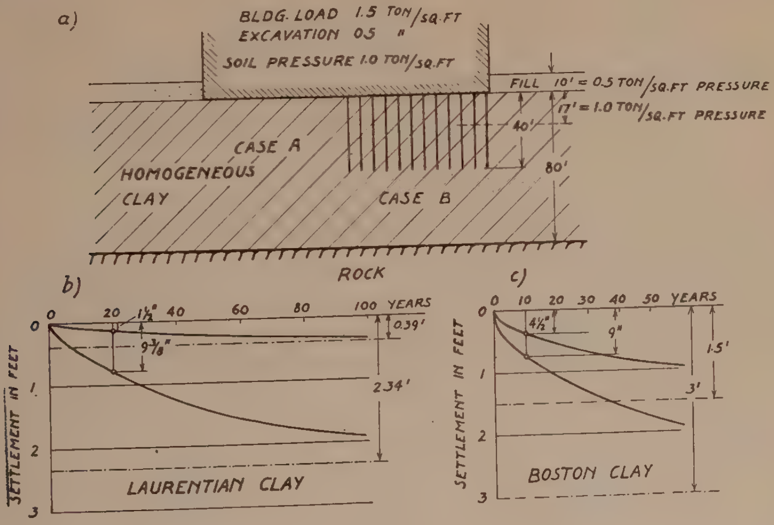


FIG. 18

In order to visualize what these figures mean, let us assume four cases in which the same building is founded on two types of homogeneous clay without and with piles, shown in Fig. 18*a*.

The total building load of 1.5 tons per square foot is reduced by an excavation of 10 feet of fill to 1.0 ton per square foot. Case *A* represents a mat foundation without piles. Case *B* represents a pile foundation with a 40-foot embedment in clay. The settlements were computed for the Laurentian clay (Fig. 11) and the Boston clay (Fig. 12), and are assembled in time-settlement diagrams in Figs. 18*b* and 18*c*.

Let us compare the settlements in these four cases *twenty years* after construction:

	Inches
Laurentian clay:	
No piles	1½
With piles	9⅜
Boston clay:	
No piles	6
With piles	12

These figures need no further interpretation. Some of you may ask how it is possible that in the case of the Laurentian clays the disastrous effect of pile-driving was not discovered before. One reason, and perhaps the only one, is the tricky nature of this clay, which succeeds remarkably well in hiding its true character beneath a layer of a very soft, sticky material into which the bottom of every excavation pit is turned by the construction activities. And the soil auger or ordinary drilling methods furnish the engineer with a clay which is equally well remolded. No wonder that the soil is considered a very bad foundation material, and that piles, the traditional cure for bad soil conditions, are called into action.

However, without the use of piles, these well-consolidated Laurentian clays will sustain any reasonable load without excessive settlements. For the Boston clay the situation is different, because also without piles the settlements are still too large, unless the building is specially designed to withstand such settlements without harm.

The remolding of the clay by pile-driving affects not only the amount of settlement, but also its rate. The rate of consolidation is determined by the coefficient of consolidation which is the quotient of the permeability and of the compressibility of the clay (9). If remolding increases the compressibility and leaves the permeability unchanged, the coefficient of consolidation will become smaller. However, the piles may facilitate the drainage of the clay on account of the fact that concrete, as well as wood in the direction of the fibers, offers less resistance to the penetration of water than the clay itself. We should, therefore, figure with a larger permeability of the clay along the piles. As a result, the coefficient of consolidation would perhaps not be much different from its value in the undisturbed state. On this assumption are based the examples represented in Fig. 18. Such a procedure is, of course, open to criticism.

The amount and the rate of compression of a clay which is remolded by pile-driving remain more or less of an estimate, as long as no further research is done; in the undisturbed state, however, these factors can be determined with a sufficient degree of accuracy.

Another problem for further investigation is the effect of the disturbance of the clay by pile-driving on the settlement of adjacent buildings. The reduction in lateral support may cause a new period of settlement for buildings which have reached already a stable position.

The economic waste of floating pile foundations in clay was first pointed out by Terzaghi, who has reached this conclusion from theoretical considerations as well as from extensive observations of the settlement of buildings founded with and without piles. (12) and (20)

EXCAVATION THE ONLY RELIABLE METHOD FOR REDUCING SETTLEMENTS

Is there a way to reduce the settlements without reducing the building load? We have employed such a method already in Fig. 18*a* without paying special attention to it. The building load is 1.5 tons per square foot, but on account of a 10-foot excavation the pressure on the clay is reduced to 1.0 ton per square foot. If the building were put right on the surface of the ground, the pressure on the clay, and, therefore, also the settlements, would be 50 per cent greater. By excavating more soil we can reduce the settlements to any desired amount. An excavation of 27 feet would lighten the load by exactly the same amount which represents the building load. That means, after construction of the building, the pressure on the clay would be the same as that under which it is consolidated. Consequently there would be no further consolidation or settlement (provided no piles were driven into the clay).*

I have herewith stated the fundamental *principle* for the design of *building foundations on clay*, where firm rock, sand or gravel are not within reach. I repeat it in a modified way:

Lighten the load by excavation so that the settlement due to consolidation of the clay will not exceed an arbitrarily fixed limit.

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* The effect of swelling of clay during the period the excavation pit is open can be neglected for two reasons: (1) the rebound curve is so flat in its first section that a reduction of 50 per cent in pressure, and sufficient time, would cause an amount of swelling which is very small in comparison with the consolidation along the compression curve (Figs. 10 and 12); (2) the period during which the load is removed is too short to cause any noticeable swelling, except in a thin surface layer.

In Fig. 19 are assembled a few typical foundation conditions and the solution which suggests itself on the basis of the preceding discussion.

Case 1. — Homogeneous clay deposit. Raft foundation without piles. Depth of excavation to be computed from the theoretical time-settlement curve and the permissible rate of settlement.

Case 2. — A soft, very compressible stratum of fill, muck, or unconsolidated clay upon a firmer clay deposit. Excavate the soft stratum down to or into the firm clay and use a raft foundation without piles; or use open caissons, with enlarged bells at their bottoms, such as Gow caissons, for transferring the load into the clay. Depth of excavation to be decided as before.

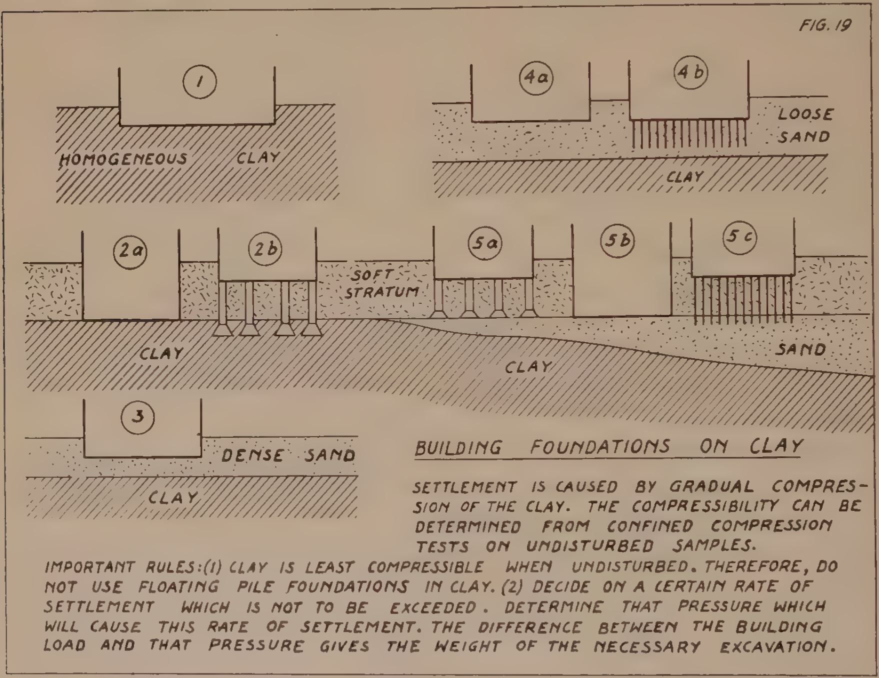


FIG. 19
①
HOMOGENEOUS CLAY
4a
4b
LOOSE SAND
CLAY
2a
2b
SOFT STRATUM
5a
5b
5c
CLAY
CLAY
SAND
3
DENSE SAND
CLAY
BUILDING FOUNDATIONS ON CLAY
SETTLEMENT IS CAUSED BY GRADUAL COMPRESSION OF THE CLAY. THE COMPRESSIBILITY CAN BE DETERMINED FROM CONFINED COMPRESSION TESTS ON UNDISTURBED SAMPLES.
IMPORTANT RULES: (1) CLAY IS LEAST COMPRESSIBLE WHEN UNDISTURBED. THEREFORE, DO NOT USE FLOATING PILE FOUNDATIONS IN CLAY. (2) DECIDE ON A CERTAIN RATE OF SETTLEMENT WHICH IS NOT TO BE EXCEEDED. DETERMINE THAT PRESSURE WHICH WILL CAUSE THIS RATE OF SETTLEMENT. THE DIFFERENCE BETWEEN THE BUILDING LOAD AND THAT PRESSURE GIVES THE WEIGHT OF THE NECESSARY EXCAVATION.

Fig. 19

Case 3. — Dense sand on clay. Raft foundation without piles. Excavation as before.

Case 4. — Loose sand on clay. Raft foundation without piles, or with piles for the purpose of compacting the sand. Excavation as before.

Case 5. — A soft, very compressible stratum (fill, muck or unconsolidated clay); below this, a layer of sand, underlaid by clay. When the soft top stratum is not too thick, it should be excavated and a raft foundation put on the sand. Should this not seem feasible, then the load must be transferred into the sand by means of piles, or, for a very thin sand layer, by means of open caissons resting with the bell on the sand. *Under no circumstances should piles be driven through the sand into the clay!* Depth of excavation should be determined as before.

Quite often the soil conditions are not as regular as shown in Fig. 19, and the resulting complications will require special studies in order to find a satisfactory solution. For a case such as that shown in Fig. 20, one has to expect much larger settlements on the right-hand side, where the sand layer is very thin and underlaid by unconsolidated clay. When such conditions are encountered, and when unequal building load distribution is not considered feasible, the only safe foundation method is to reduce the magnitude of the settlements to a very small quantity, or to zero, by means of excavation.

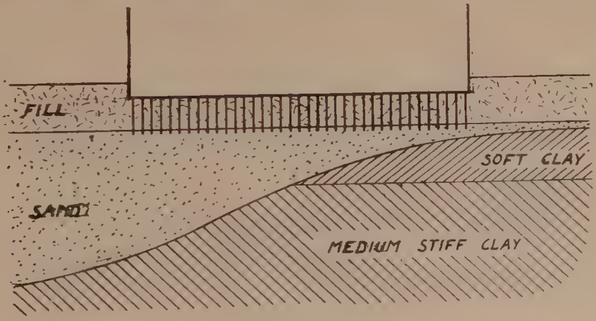


FIG. 20

EXCAVATION MAY BE EXTENDED LATERALLY AND USED AS INSULATION OF BUILDING AGAINST VIBRATIONS

The depth of the necessary excavation may often be reduced by a lateral extension beneath the sidewalks, as shown in Fig. 21. This design allows a very fortunate incorporation of an insulation trench for eliminating the transmission of vibrations into the building. This excellent insulation against vibrations which such an arrangement insures has been demonstrated by research carried out in Germany during the past few years. As a result, several new office buildings in German cities have incorporated this feature in their design.

TRUSS FOUNDATIONS NECESSARY TO INSURE RIGIDITY

The settlements of a building founded on clay with individual footings is never uniform, not even on perfectly homogeneous ground. On the other hand, the superstructure is not designed for such deformations. In fact, the design of the superstructure is based on the assumption that no differential settlement will occur. If in many cases the effects of non-uniform settlements do not seem to harm the buildings, it is due to the slow rate of subsidence which allows the structure to adjust itself. Besides, even if the building is not suffering to any great extent, there is continuous trouble with the cracking of the pipe lines and the scaling of the plaster, etc. In all cases, where non-uniform settlements proceed at a fast rate, the buildings themselves may be seriously harmed.

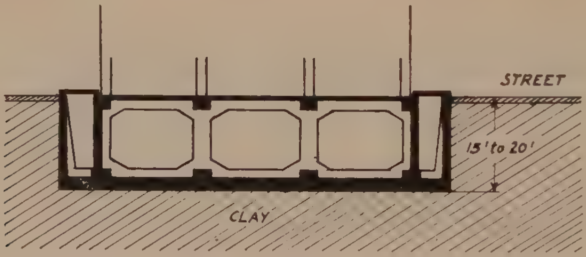


FIG. 21

In my opinion a foundation must be designed so that for any possible stress distribution no harmful differential movements can occur in the same building. This does not mean that the building must not settle, but it means that the foundation should be rigid enough to prevent any appreciable bending. A foundation slab alone cannot accomplish this in an economical way. In order to make a raft foundation rigid enough, and at the same time not excessively heavy, it must be designed as a truss, combining as its members the whole basement. The system which seems best suited for this purpose is the Vierendeel truss, consisting of rigid frames (Fig. 21), as it was for the first time applied in the Telephone Building in Albany, New York (Fig. 27).

In this connection arises the question of the distribution of soil reaction beneath a rigid foundation, or the reciprocal problem of the curve of deflection beneath a uniformly loaded area. Both depend entirely on the elastic properties of the soil.

As long as the modulus of elasticity of the soil is constant, that is, independent of the magnitude and the ratio of the principal stresses, we may apply the equations which Boussinesq has derived for the stress distribution and the deflection beneath a point load (24). The integration of these equations for arbitrary loaded areas is rather involved, and one must often be satisfied with approximate solutions. Such computations show that the deflection beneath a uniformly loaded area is as shown in Fig. 22*a*, with a maximum at the center of the area. The stress distribution beneath a rigid area is represented in Fig. 22*b*. The stress has a minimum at the center of the area and is, theoretically, infinitely large at the edge. Practically the distribution is as shown by the full-drawn curve in Fig. 22*b*, on account of the fact that Hooke's law is not valid in the vicinity of rupture, for any material.

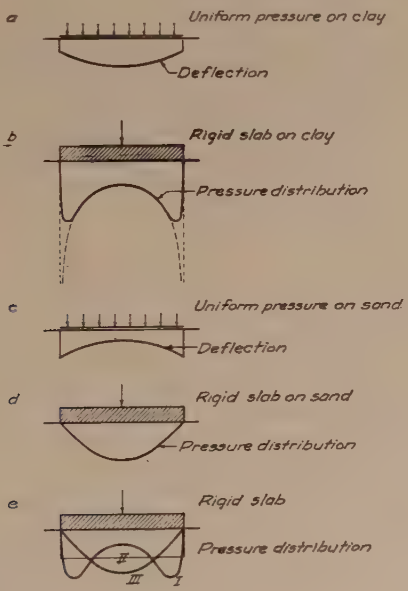


FIG. 22

Soils which are sufficiently elastic so that Figs. 22*a* and 22*b* would apply, are undisturbed clays of the type of the Laurentian clays or the Albany clay. As consolidation of the clay proceeds under the building load, the stress distribution beneath a rigid foundation seems to change gradually and to become more uniform.

That the stress distribution on clay and similar soils beneath rigid areas must have a maximum near the edges, can be concluded indirectly from settlement observations on uniformly loaded areas which assume a dished shape. Striking examples are discussed by Mr. Lazarus White in (14). However, no experimental evidence is as yet available on the stress distribution in clays.

For most soils, especially for cohesionless sand, the modulus of elasticity is not a constant, but increases about proportionally with the smaller principal stress. As a result, the soil beneath the central part of a loaded area, which is confined by a large horizontal stress component (minor principal stress), undergoes a smaller deformation than in a perfectly elastic material. Therefore the deflection beneath a uniformly loaded area becomes a maximum at the edges and a minimum beneath the center, as shown in Fig. 22*c*. The stress distribution beneath a rigid area has a maximum at the center and a minimum along the edge, as shown in Fig. 22*d*. Extensive experimental investigations on the deformation and stress distribution beneath loaded areas on sand were carried out by Strohschneider, Enger, Goldbeck, Kögler and Scheidig. An excellent summary of these investigations, with references, is contained in the publications by Kögler and Scheidig (15) and (16). Scheidig refers, also, to observations on buildings founded on sand which have settled in accordance with the results of small scale tests shown in Fig. 22*c*. (16) The settlements at the ends of the buildings were so much greater that serious cracking of the walls developed.

We arrive at the final conclusion that the stress distribution beneath rigid areas may be either concave or convex, depending on the elastic properties of the soil, and that the stress distribution on clay and similar soils may undergo considerable changes in the course of time. In order to be absolutely safe, the design of a truss foundation should be based on the extreme possible distributions, represented by curves I and III in Fig. 22*e*. At first consideration it appears too costly to base the design of the truss on so widely different stress distributions. In reality, a somewhat heavier reinforcement in some sections of the upper and lower chords of the truss is sufficient to assure safety against any distribution of soil reaction and its change with time. Such a procedure is especially desirable where sand and clay layers alternate, or when older adjacent buildings have caused consolidation of the soil along one or several sides of the new building.

A non-rigid foundation mat or individual spread footings may be used when the total settlement will not exceed about one inch. Such cases are foundations on rock, or on gravel and sand which are not underlaid by compressible soil layers.

CORROBORATIVE EVIDENCE FROM OBSERVATIONS ON BUILDINGS

I consider myself very fortunate that I have had at my disposal observations on existing structures which support my contention that piles driven into undisturbed clay increase the settlements.

The first case is represented by a group of buildings founded on Boston clay. In Fig. 23 is assembled all information pertaining to our problem, that is, the distribution of load, the soil profile, the depth to which the piles are driven, and the settlements at different periods after construction.

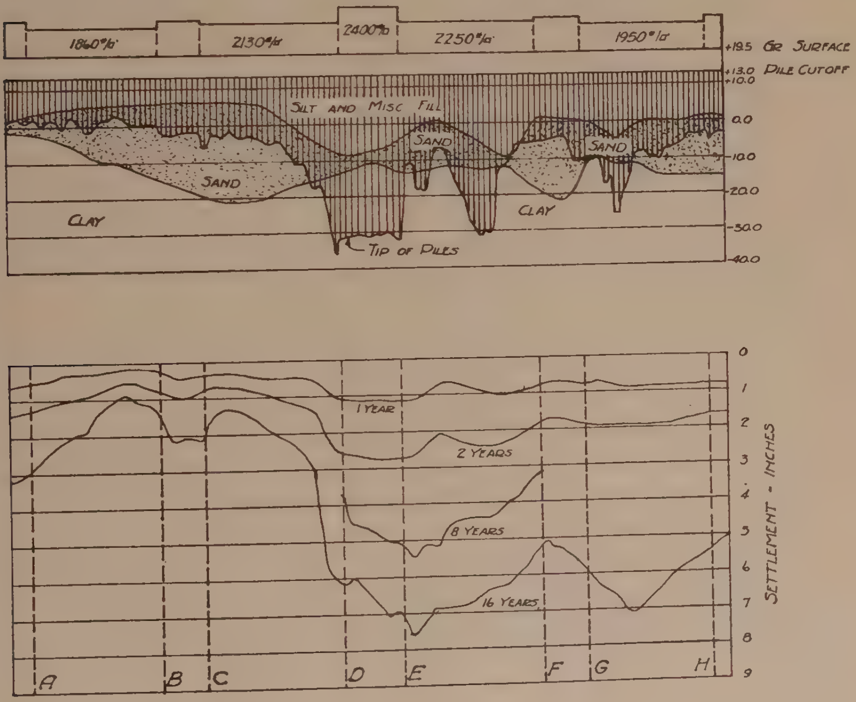


FIG. 23

The clay is fairly homogeneous and approximately of uniform compressibility, except for a very soft layer above elevation —10 at the extreme left. The thickness of the compressible clay layer varies between 55 and 85 feet.

Taking into consideration the differences in thickness of the clay layer and the differences in load, the settlement curve after sixteen years should show much less variations, and the maximum settlement should be smaller than is represented by the actually observed settlement curve. Due to the disturbance of the clay structure, the settlement is much larger in those sections where the piles penetrate into the clay. The effect of the disturbance spreads a short distance beyond the region of the disturbance, so that the undisturbed regions between zones of disturbance show also larger settlements. This is partially due to a decrease in lateral support of the undisturbed clay mass.

The considerable settlement on the extreme left is due to the less consolidated clay above elevation —10 and a slight load concentration.

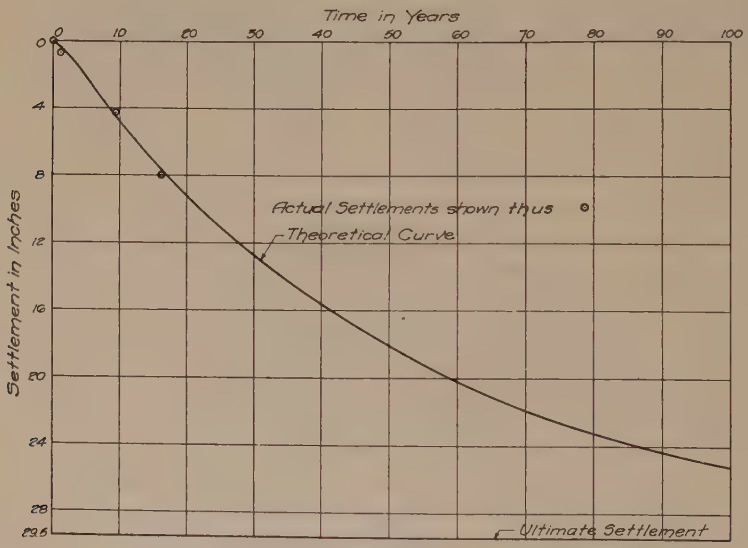


FIG. 24

The depression between sections *B* and *C* is due to a load concentration. The sudden increase of subsidence to the left of section *D* can only partially be explained by the larger load. The fact that the sudden drop coincides with the border line of undisturbed and disturbed soil, and not with section *D* where the load increase starts, is a remarkable evidence for the disturbing effect of piles.

Some inconsistencies, when comparing the settlements during the first year with those during a subsequent period, are due to the fact that we do not know the accurate amount of settlement during the first months of construction. The actual settlement during the first year is probably much greater than shown in Fig. 23.

The second example is represented by an extensive study of the causes of the settlements of an existing building which was carried out by Professor Gilboy (17). Fortunately, the report on this investigation will soon be available in published form, so that I can refrain from going into detail and just state those results which pertain to our discussion.

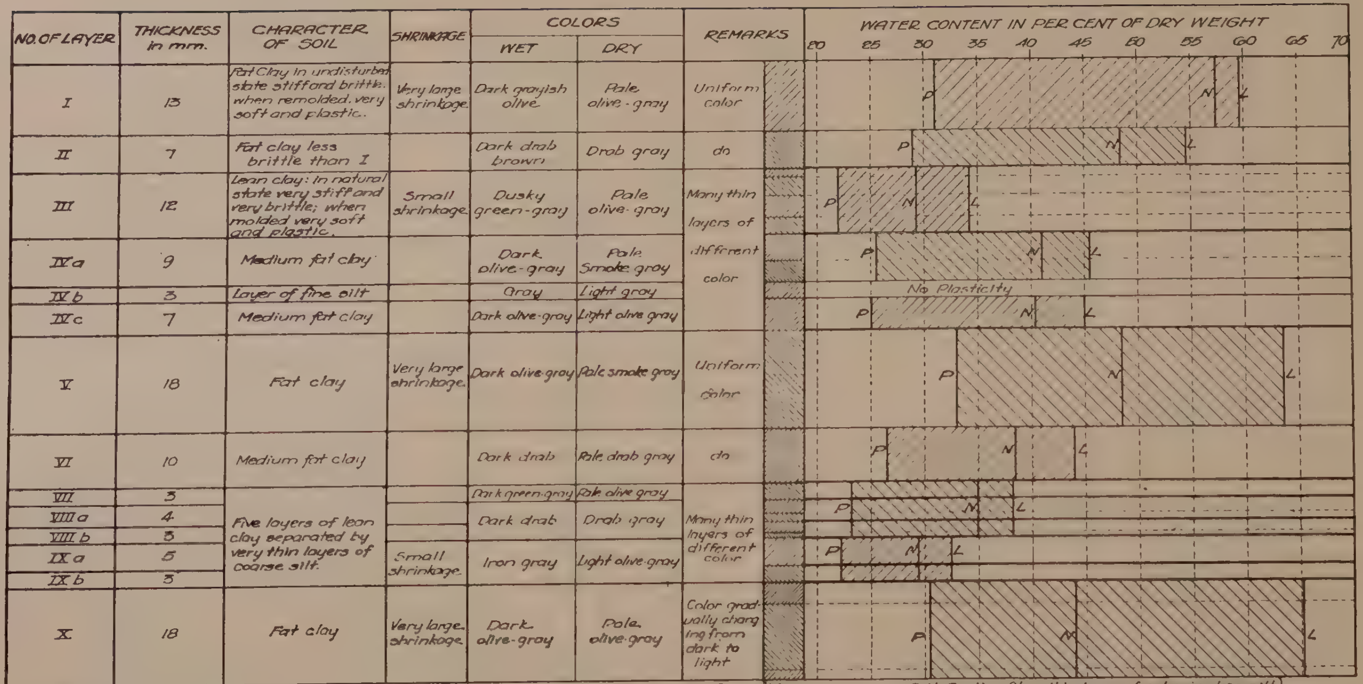
The object of this investigation was a building with a pile foundation on Boston clay. The embedment of the piles in the clay is between 20 and 25 feet. Undisturbed soil samples were taken from different elevations, and compression tests were carried out on all the samples in the undisturbed and remolded states. In the computation of the theoretical settlements the remolding of the clay was taken into consideration. A comparison of the theoretical and actual settlements indicates a very good agreement. The actually observed settlements for the present time vary between 6.5 and 8.5 inches, and the computed settlements between 6.0 and 8.2 inches. The progress of the subsidence with time is shown in Fig. 24. The circles represent the observed values, and the curve represents the theoretical relation between time and settlement. Fifty years after construction the settlement will be 1½ feet. The total settlement will increase in the course of about two hundred years to a value of 2½ feet.

The question arises whether, in this case and the one cited before, the settlements would have been reduced to a tolerable amount if the clay had not been disturbed. The reduction of the settlements would be between 30 and 50 per cent; that means the settlements would still be considered too large. Consequently, one would have to lighten the load by an appropriate excavation, or transfer the load to rock by means of one of the modern methods (open caissons, steel piles, etc.).

In discussing these cases, there is no intent to criticize the designers. At the time when these buildings were planned almost nothing was known about the causes of the settlements of structures founded on clay.

As the third example, I have chosen a case which, in my opinion, represents a well-designed foundation. It is the Albany Main Telephone Building in Albany, New York. (21)

The Albany clay, on which this building is founded, is a nonhomogeneous deposit consisting of very thin layers of highly plastic clay, alternating with layers of silty clay, cohesionless silt and very fine



NO. OF LAYER	THICKNESS in mm.	CHARACTER OF SOIL	SHRINKAGE	COLORS		REMARKS
				WET	DRY	
I	13	Fat Clay in undisturbed state stiff and brittle; when remolded, very soft and plastic.	Very large shrinkage.	Dark grayish olive	Pale olive-gray	Uniform color
II	7	Fat clay less brittle than I		Dark drab brown	Drab gray	do
III	12	Lean clay: in natural state very stiff and very brittle; when molded very soft and plastic.	Small shrinkage	Dusky green-gray	Pale olive-gray	Many thin layers of different color
IV a	9	Medium fat clay		Dark olive-gray	Pale Smoke gray	
IV b	3	Layer of fine silt		Gray	Light gray	
IV c	7	Medium fat clay		Dark olive-gray	Light olive gray	
V	18	Fat clay	Very large shrinkage.	Dark olive-gray	Pale smoke gray	Uniform color
VI	10	Medium fat clay		Dark drab	Pale drab gray	do
VII	3	Five layers of lean clay separated by very thin layers of coarse silt.		Dark green-gray	Pale olive gray	Many thin layers of different color
VIII a	4			Dark drab	Drab gray	
VIII b	3					
IX a	5		Small shrinkage.	Iron gray	Light olive-gray	
IX b	3					
X	18	Fat clay	Very large shrinkage.	Dark olive-gray	Pale olive-gray	Color gradually changing from dark to light

Notes:
P · Plastic Limit
L · Liquid Limit
N · Water content in natural state.

Shaded areas between lines marked P and L represent plastic range.

—— Silt Parting (Very thin layer of cohesionless silt)
—— Sudden change in degree of plasticity, color, or both.
- - - Gradual change in color

FIG. 25

sand. The variations in water content and plasticity of an undisturbed block of 4½ inches total thickness are shown in Fig. 25.

Without going into a discussion of the physical meaning of the plastic and liquid limit of clay (11), I wish only to point out that the larger the difference between these two limits, the more plastic and the more compressible is the clay. For comparison I call your attention to layers VI to IX as being most nearly like the Boston clay. About one-third of the material is twice as plastic, and in remolded state approximately twice as compressible, as the Boston clay. The remainder compares closely with the Boston clay. The total thickness of the cohesionless material is small. However, its importance from the standpoint of rate of settlement is tremendous, as was first pointed out by Professor Terzaghi. Each sand and silt layer represents an excellent means for draining off the water which is being squeezed out of the clay under the building load, provided that these layers have, somewhere, a free outlet. For perhaps the greater part of the depth of this clay such outlets actually exist. Consequently, a part of the total compression will occur at a very fast rate, starting during construction. (Fig. 26, curve *A*.) The balance will follow a consolidation curve similar to that for the Boston clay. As a result, we have a time-settlement curve which shows a very large subsidence during the first year, and then leads over to a very slowly progressing settlement. The actual shape of the time-settlement curve could be determined only on the basis of extensive field and laboratory investigations.

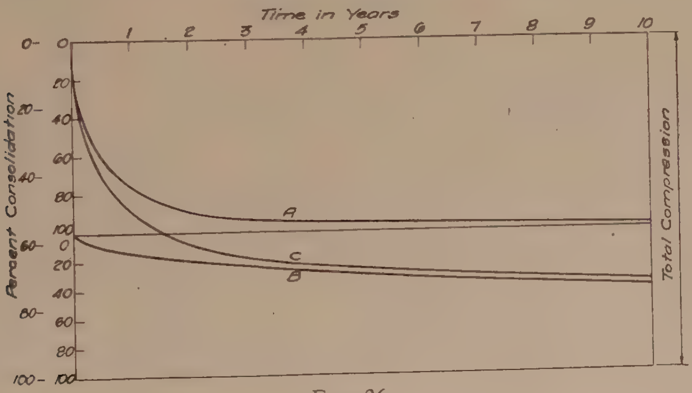
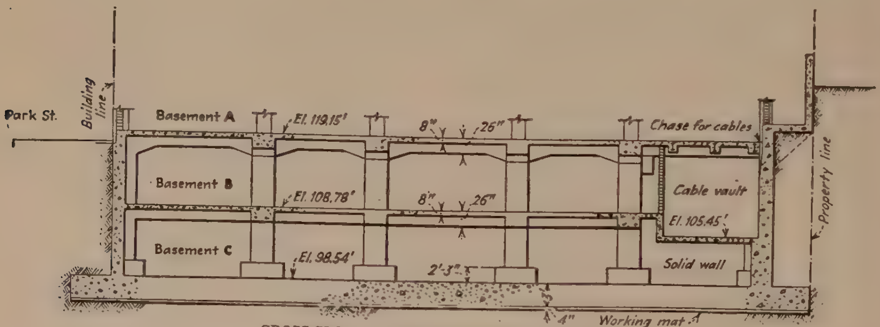


FIG. 26

The structural effect is very pronounced in the Albany clay. In the undisturbed state this clay appears brittle, and it is considerably harder than Boston clay. When remolded it becomes very soft and sticky. These characteristics are nearer to the Laurentian clays than the Boston clay. The difference in compressibility between the undisturbed and the disturbed state is probably much larger than for the Boston clay, but smaller than for the Laurentian clays.

The fast rate of settlement and the large increase in compressibility of that portion of the clay which is disturbed by pile-driving account for the serious settlements which were observed in Albany, especially on some newer buildings on pile foundations.

The design of the foundation which was adopted for the Telephone Building after careful studies is shown in Fig. 27. It represents a truss of the Vierendeel type, which is designed so strong that it can resist any distribution of soil reaction without rupture or any appreciable deformation. The total building load of 4,500 pounds per square foot is reduced to about one-half by an excavation of 20 to 25 feet of soil.



CROSS-SECTION OF RIGID FRAME FOUNDATION

FROM ENGINEERING NEWS-RECORD, NOV. 27, 1930. ALBANY TELEPHONE BUILDING

FIG. 27

The progress of settlement with time and its relation to the loading of the ground is represented in Fig. 28, which is based on data supplied through the courtesy of Mr. W. H. Mueser. No settlements occurred during construction as long as the load was less than the weight of the excavation. As soon as this point was exceeded the structure began to settle. The shape of the time-settlement curve indicates that 80 per cent consolidation was reached within about four months after application of the load.

For one-half of the building area the settlements average about 1¼ inches, with a variation of ¼ inch. Fig. 28 represents a typical settlement curve for this section of the building. Toward the north end of the building the settlements increase gradually to a maximum of 2½ inches.

From Fig. 28 we can estimate that the settlement of the building, if it were placed directly on the ground surface, would be about three times greater, that is, between 4 and 7 inches. If, in addition, the clay were remolded by pile-driving, we should have to anticipate settlements of at least 10 inches during the first year after construction. Such large settlements may not be harmful to a building on a well-designed truss foundation, but they would certainly cause very serious damage when founded on individual footings, on account of the inevitable large variations in settlement in different parts of the building.

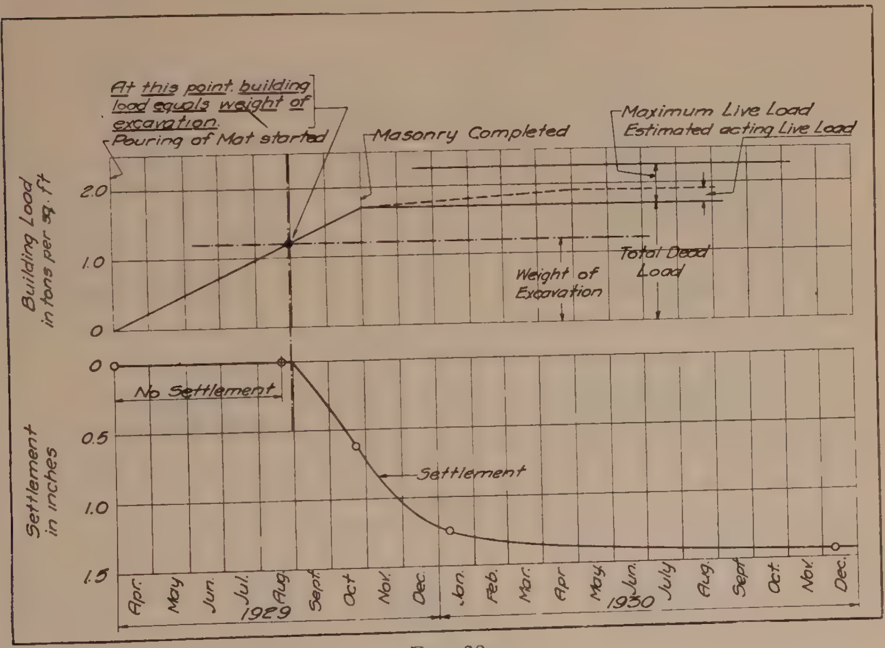
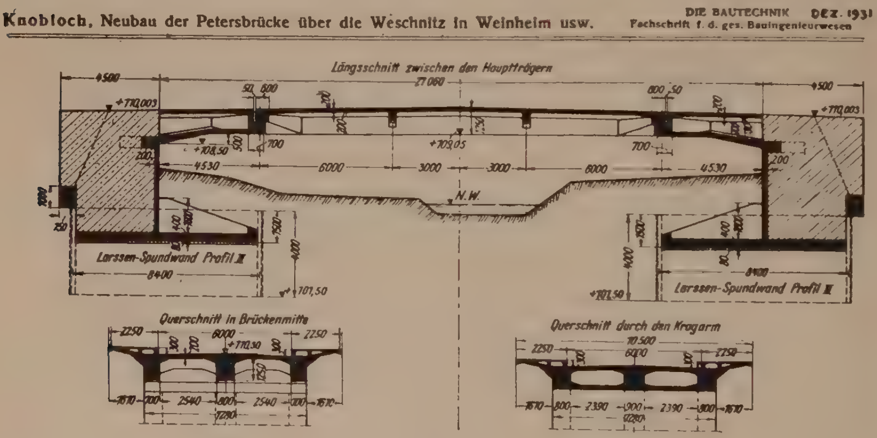


FIG. 28

I have more information from other places which corroborates the conclusions drawn from our research work. In part, this information is qualitative and should be supplemented by quantitative data before making it public. Other data I have received under the condition of keeping them confidential. Business interests and a false feeling of shame regarding unsuccessful foundations are the two main reasons why such cases are often kept secret instead of being made the object of careful investigation for the purpose of understanding the causes of excessive settlements. Records of other interesting settlement observations are included in the publications of Professor Terzaghi, particularly (18), (19) and (20).

Note on Unsymmetrical and Concentrated Loading. — As stated in the introduction, the design of foundations for concentrated or unsymmetrical loads is often determined by the safety against rupture of the soil. For this reason piles are sometimes the only means of protection. This applies especially to harbor construction (Fig. 1).



Knobloch, Neubau der Petersbrücke über die Weschnitz in Weinheim usw. DIE BAUTECHNIK Dez. 1931 Fachschrift f. d. ges. Bauingenieurwesen

Längsschnitt zwischen den Hauptträgern
27 060
4500
+710,003
4530
6000
3000
3000
6000
4530
+709,05
+708,50
700
200
N.W.
Larssen-Spundwand Profil II
8400
+701,50
Querschnitt in Brückenmitte
Querschnitt durch den Kragarm
2250
6000
2540
10 500
2390

Fig. 29

However, in view of the fact that the driving of ordinary piles, especially in groups, has the serious disadvantage of increasing the settlements of the structure, one should try to use steel sheet piles for such foundations. An excellent example is shown in Fig. 29. (23) The abutment of this bridge is founded on homogeneous clay of such a character that a pile foundation would have been used by most engineers. However, the design in Fig. 30 is by far superior. The steel sheet piling surrounding the spread footing reduces the settlement due to lateral deformation of the clay and makes the foundation safe against rupture of the soil. The clay below the foundation remains undisturbed, and therefore will undergo a minimum of settlement if compared with other possible designs of this foundation.

Further Research Necessary for Devising Simplified Procedures

The last question which I should like to touch upon briefly is that of the possibility of determining the settlements on clay by simpler methods than by expensive core borings and elaborate testing of the samples. The most obvious idea is that of loading tests. I thought for some time that it should be possible to derive somehow, from loading tests on clay, the settlements which a building will undergo. However, I have definitely come to the conclusion that this is not possible. We may from such tests get an idea of the strength of the clay, and we can very well decide whether a clay layer is consolidated enough to support a building without excessive lateral deformation, but we cannot tell what the settlement will be, nor at what rate it will proceed. Two soils may give identical results in loading tests, while under the same building load one may settle 3 inches and the other 20 inches within the first ten years.

To discuss this matter properly would take too long, and I must satisfy myself with a short explanation. The compression of the clay under a small area affects the clay mass to a proportionally smaller depth. If a building settles one foot in the first ten years, it may correspond to a compression due to volume change of about one-quarter inch for a load test on one square foot (Fig. 30). However, even if the loading test were continued with utmost accuracy over many months or a whole year to get part of the consolidation curve, and if we should succeed in solving the difficult problem of three dimensional consolidation, we should still have to eliminate the unknown portion of settlement due to lateral deformation.

In the case of the building load the relative importance of the lateral deformation, if compared with the compression, is small as long as the clay is at all consolidated enough for bearing loads. For the purposes of building construction it is sufficient to assume that the settlement due to compression is the whole settlement which is to be expected. However, in loading tests the larger portion of the settlement is due to lateral deformation. For this reason the compression under a large area cannot be derived from a small-scale loading test.

Loading tests on piles resting in clay inform us about the maximum load which a pile or a group of piles will carry without movement relative to the soil.* But regarding the compression of the soil itself, and the settlements which a building will undergo in the course of time, pile loading tests do not yield any information whatsoever. This point was brought out clearly by Terzaghi in (12), where he concluded: "The essential part of the problem consists of studying the effect of the load on the ground located around the piles and beneath the points. Yet building codes are satisfied with specifying the load per pile, disregarding the true cause of future complications." A pertinent treatment of this subject is also contained in a recent paper by J. S. Crandall. (25)

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* Load tests on piles are of special importance in underpinning work. (22)

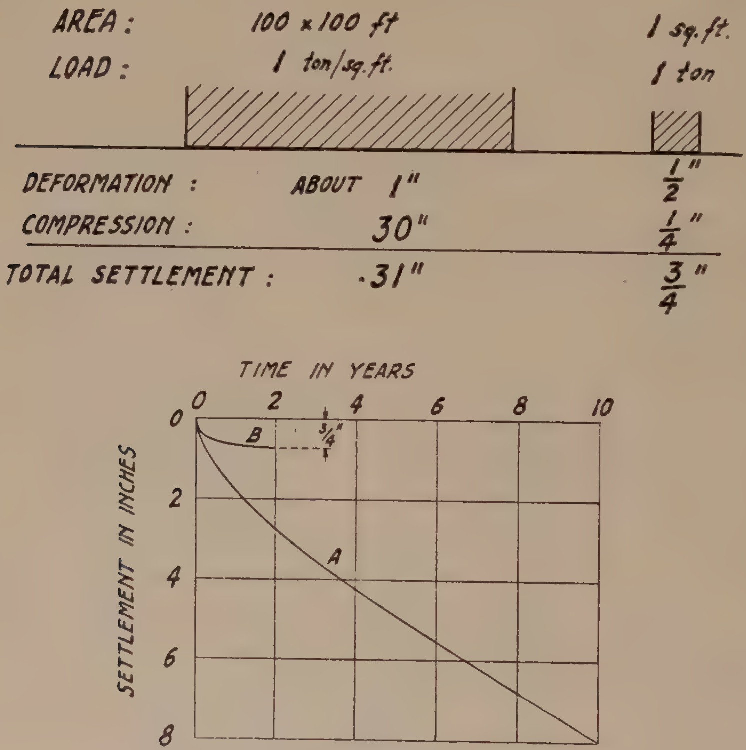

Fig. 30

It is reasonable to expect that simple methods other than loading tests will eventually be developed which will provide for an approximate solution of the problem without resorting to the rather elaborate tests and computations now required. Such development would have to be based on a thorough exploration of the underground of a particular locality, and a subsequent correlation of the consolidation characteristics of the clay with the results of some simpler test procedure. Success along this line can be expected only when engineers realize the importance of such studies and co-operate in the establishment and support of research organizations in various parts of the country.

Conclusion

I have tried to illustrate that the whole problem of building foundations on clay boils down to these *two* simple *principles:* first, *do not disturb the natural structure of the clay; if you do, no human being is able to restore its original strength;* second, *decide on a certain rate of settlement which you do not wish to exceed, and determine that pressure which will cause this rate of settlement; the difference between the building load and the above pressure is the weight of soil which must be removed before erecting the building.*

A definite bearing value of clay does not exist. As long as engineers are guided by building codes containing definite bearing values for clay, they are consciously or unconsciously guessing without any assurance in their own minds that they are guessing correctly.

The engineer must learn that the kind of questions he asks an expert regarding the properties of a clay underground should not be, "How much load may I put on this soil?" Or, in an apparently more scientific manner, "What is the bearing capacity or the bearing value of this clay?" His question should be, "How must I design my foundation so that the rate of settlement under the given building load will not exceed certain limits?"

After the engineer has obtained this information, it is up to him to decide if such a foundation is feasible from a financial standpoint. With the increasing efficiency of modern foundation methods* the possibility of transferring the loads to rock or firm soil in great depths has to be considered in many cases where, not long ago, such a procedure would have been prohibitive in cost.

I realize that I have presented to you ideas which are radically different from traditional engineering practice, and on this account I am expecting many unfavorable reactions. But I do not expect any one to believe what I have said without first convincing himself of the truth by his own studies. This conviction will come only by studying the evidence from field observations such as that presented to you in this lecture and contained in the forthcoming report by Professor Gilboy; by careful personal observation of all factors influencing the settlement of structures; and by keeping in close touch with the experiences of other engineers. *From the standpoint of progress, it is of utmost importance to analyze unsuccessful foundations and to bring the facts to the attention of the whole engineering profession by unrestrained publication.*

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* Chicago well method, different types of open caissons, steel tube piles, etc.

ACKNOWLEDGMENTS

The researches described in this lecture were carried out at the Massachusetts Institute of Technology as part of a large program in which the Institute is co-operating with the United States Bureau of Public Roads and with the American Society of Civil Engineers. In particular, the support of Professor Spofford, Head of the Civil Engineering Department, and the hearty co-operation of Professor Gilboy and the staff of the Soil Mechanics Laboratory have made possible the performance of valuable research in foundation and highway engineering.

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Discussion

Charles M. Spofford:* This paper is a notable addition to the literature on foundations and is worthy of the thoughtful consideration of all engineers responsible for the construction and design of foundations bearing upon clay. The fact that laboratory tests show that some clays lose much of their resistance to compression by being remolded is of much importance, and the explanation presented by the author for this reduction is interesting and plausible, even if not supported by rigid proof.

The hypothesis that the driving of piles into clays similar to those tested in the laboratory reduces their compressive resistance in the same manner as has been found to occur in the remolding of laboratory specimens seems reasonable, and offers an explanation of the greater settlement which has been observed in certain cases of structures supported on piles as compared with structures supported upon a continuous foundation bearing upon similar soil. The writer hopes that the Soil Mechanics Laboratory at Technology will, in time, find it possible to obtain conclusive data upon the condition of clay which has been disturbed by pile-driving and by the jacking of piles as is done in the installation of pre-test piles in order to test the validity of the foregoing hypothesis.

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* Hayward Professor of Civil Engineering, and in charge of Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, including the Soil Mechanics Laboratory; Consulting Engineer, Fay, Spofford & Thorndike, Boston, Mass.

While the method to be used for distributing the weight of any given structure over a clay foundation must continue to be based on economical factors determined both by the original cost and the cost of upkeep of the structure, its correct determination involves the securing of as complete data as possible upon the probable magnitude, uniformity and rate of settlement, and the researches being conducted by the author and the others connected with the staff of the Soil Mechanics Laboratory are, in the writer's judgment, adding greatly to the data available for making such decisions.

In conclusion, the writer wishes to congratulate the author of the paper upon the thoroughness of his investigation and the clearness with which he has presented the subject, and to express the hope that it will be freely criticized by all who may hear or read it in order that all possible phases of the problem may be brought to the attention of the author and receive his careful consideration.

FRANK A. MARSTON:* The more experience one has in the examination and testing of clay soils, the more impressed one is with the need for research regarding the characteristics and behavior of those soils, composed entirely of very fine grains. Grain size alone is not necessarily a controlling factor from which the behavior of the material can be predicted. For comparing samples of clay from various sources, grain size may be quite misleading as a means of determining other characteristics. This being true, it is no wonder that judgment based solely on visible characteristics has, at times, been found unreliable even in the case of experienced observers.

The shape of the grains has a distinct effect on such characteristics as compressibility and plasticity. This has been demonstrated by experimenting with powdered minerals that break up into scale-like particles, as, for example, tests by Dr. Glennon Gilboy † on the compressibility of mixtures of sand and mica, the sand being composed of bulky grains of uniform size, and the mica of scale-like grains of similar size.

From such tests the indications are that clays containing a large percentage of scale-like particles, like bentonite, will be found to be highly plastic, very compressible and quite elastic.

Soils formed by deposition in water vary greatly in structure, depending on the rapidity with which they were deposited and the manner of consolidation. An interesting illustration of this is the relative consolidation of ocean beach sands.

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* Partner, Metcalf & Eddy, Boston, Mass.

† Proceedings of American Society of Civil Engineers, February, 1928.

At Daytona Beach, Florida, well known for the hardness of the beach surface when exposed at low tide (permitting automobile speed tests), the sizes of sand grains were much the same as on the beach one mile above Ormond, Florida, where the surface was too soft to permit automobile traffic. The chief difference in these two sands appeared to be in the volume of voids, the softer sand mass having the larger percentage of voids.

Recently I examined a trench about 30 feet deep and 10 feet wide which was being excavated in blue-gray clay by means of a heavy clamshell bucket operated from the trolley of a trench machine. When the bucket was dropped from a height of 5 feet on to the clay bottom there was a ringing sound from the impact, indicating the compact nature of the clay. The sound was not unlike that when such a bucket strikes soft shale rock. When this clay was disturbed, however, it became soft, very wet, sticky and difficult to walk in. Apparently it was of a structure of considerable compressive strength until disturbed, and then it was completely altered as an excess of water was released.

Compression tests on clays in an undisturbed state, described by Mr. Casagrande, as compared with tests on the same clays after having been remolded, show much smaller changes in volume in the undisturbed state.

Apparently certain clays have a form of structure that permits of a high void ratio and still offers considerable resistance to changes in volume. When, however, this structure is broken down, the water filling the voids is released and the once stiff clay becomes very soft and plastic. One sample in particular, a Laurentian clay from Canada, exhibited this characteristic to a remarkable degree.

It would be interesting to know if Mr. Casagrande has found a clay in which there was but little difference in consolidation between the undisturbed and disturbed samples.

If the characteristic above referred to is dependent on a sort of "honeycomb" type of structure, it is reasonable to expect that some clays will show it to a much greater degree than others.

Have tests been made to show a difference in consolidation, if pressure is applied perpendicularly to the bedding plane of the Laurentian clay and in a line parallel with the plane?

Has an analysis been made of the Laurentian clay to see whether it is composed largely of constituents breaking down into scale-like particles as was done with bentonite by Messrs. Ross and Shannon?*

Do the results of the tests with Laurentian clay indicate a confirmation of Dr. Terzaghi's hypothesis that the compressibility and elastic rebound of soils seem to be controlled by the greater or smaller abundance of scale-like particles?* Or is it possible that there may be clay composed largely of bulky grains arranged in a structure of the honeycomb type that will exhibit a great variation in compressibility, depending on whether or not its structure is disturbed?

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* Journal of American Ceramic Society, Vol. 9, February, 1926.

In closing, the writer wishes to express his appreciation of the valuable paper Mr. Casagrande has presented.

J. STUART CRANDALL:† Mr. Casagrande has described some of those physical characteristics of clay which are of vital interest to engineers. The marked difference between the rates of settlement of natural and remolded clay is of great importance in foundation design.

Building codes very definitely limit the allowable loads on various soils, but are extremely vague as to the classification of these soils. Furthermore, in clays the allowable loads do not take into account varying rates of settlement or the thickness of the beds. The suggestion that allowable loads be based on a limited settlement for the entire structure is a most logical one and merits serious consideration.

There is a provision in some building codes permitting the unit load intensity on soil underlying the foundation to be calculated as if distributed through the overlying bed at an angle of 60° with the horizontal. For isolated structures this procedure may be permissible. For closely adjacent structures, as in cities, it is a false assumption, for the soil at either side is stressed by the structure over it. Therefore it seems more logical to require that the unit load on any substratum be considered the same as that directly under the foundation.

The relationship between the results with remolded clay and piles driven in clay is very significant, and apparently explains in part the frequently observed discrepancy between the rate of settlement of individual piles under test and that of an entire structure. This discrepancy is due in part, also, to the marked difference between the stresses in the soil produced by a single pile and those produced by a group of piles. In designing pile foundations for any soil, it would seem logical to require that the stress in the soil at the level of the points be considered the same as the unit intensity directly under the structure, and that the allowable unit load be chosen on the basis of a limited settlement over a given period, say one year. For piles driven in clay, it would seem reasonable to limit the allowable load on the soil at the level of the points to a fraction, possibly one-half for Boston clays, of the normal allowable load in order to compensate somewhat for the accelerated settlement due to the remolding of the clay during driving.

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* Principles of Final Soil Classification. Terzaghi. Public Roads, May, 1927, p. 42.

† The Crandall Engineering Company.

In conclusion, let me recall an excellent editorial on "Building Codes" in the "Engineering News-Record" of November 13, 1930, of which the following is an excerpt:

> Their [Building Codes] prescriptions are usually narrow, partly with a view of enabling men unskilled in building science to apply them, and partly with a view to facilitate control by a highly superficial system of inspection on the part of the public building authority. They are not only narrow but rigid. . . . It leads to the further question whether it is not appropriate to the demands of present times to turn rather decidedly away from fixation and seek to make building codes invite new materials and methods rather than discourage them.

Mr. Casagrande's research indicates the need for new methods for determining the allowable loads on clays. It is the task of engineers to collaborate in the development of these methods and in putting them into effective use.

Irving B. Crosby:* Mr. Casagrande has given an excellent discussion of the physical properties of clay. The knowledge obtained by these researches on clay has important applications, some of which Mr. Casagrande has described, and I wish to discuss a case in which both soil mechanics and geology were used to solve a difficult and important problem.

There is a close relation between geology and soil mechanics. Geology deals both with hard rocks and with unconsolidated deposits, such as clay, sand and gravel, while soil mechanics deals only with the latter. Part of the study of the unconsolidated sediments is strictly in the field of geology, part is in the field of soil mechanics, and part is on the border line. The deposition, arrangement and alteration of sediments are strictly in the field of geology. The movement of water through the ground is studied both by geologists and by soil physicists. Much early work in this line was done by geologists and published by the United States Geological Survey. Engineers also made important contributions. In recent years soil mechanics has developed as an independent science, but one that is closely related to geology, physics and engineering, and great progress has been made.

It is impossible to draw a sharp line between geology and soil mechanics, either in theory or in practice, and the best progress in the study of the unconsolidated deposits can be made by the co-operation of geologists and soil physicists. The advantage of co-operation has been recently demonstrated by a joint study of clay at a place where a high clay bank was loaded with important buildings and where there had been slides. The investigation involved a thorough field study by the geologist to determine everything possible about the arrangement and location of the clay layers; then samples were studied and many laboratory tests were made by the soil physicist; and finally all the results, both from the field and the laboratory, were jointly studied and final conclusions were reached. It is certain that as satisfactory results could not have been reached by either working alone.

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* Consulting Geologist, 6 Beacon Street, Boston, Mass.

The clay was unusually hard, and it was difficult to understand why slides occurred in this clay under conditions which would probably not cause slides in softer clays. The geological study showed that the clay was intersected by regular systems of parallel cracks similar to the joint cracks in hard rocks. The slides are believed to have been caused by hydrostatic pressure in the cracks. By careful drainage of the surface the supply of water to these cracks can be cut off and the danger of slides reduced. These joint cracks in the clay were found not only in the steep bank, where it might seem that they could have been formed by local movements, but they were also found in places far removed from any steep slope. It is possible that these cracks were formed by earthquake vibration.

Mr. Casagrande has spoken of the great difference in the characteristics of the undisturbed and the disturbed clay, and that difference was apparent in the study just mentioned. The clay in this case was almost as hard as shale when undisturbed, but could be worked into a soft, plastic mass. The undisturbed clay appeared to be a good foundation material, but it is susceptible to sliding under special conditions. When it has been disturbed, this clay is a very poor foundation material. Many old slides have occurred at this place, and there are great masses of disturbed clay. Buildings have been built on this clay, and great difficulties were encountered with the foundations. The contrast between the behavior of the two clays was very marked; yet they were originally the same.

Mr. Casagrande has spoken of the curved sliding surfaces of clay slides, and these appear to be somewhat analogous to the curved sliding surfaces of large faults in hard rocks.

Many opportunities remain for co-operation between the geologist and soil physicist in scientific research as well as in practical problems. The microscopic study of clays is a geologic problem, and much remains to be done in this field. It should be possible to analyze the structure of undisturbed clay, and, by correlating the facts learned with the physical properties, to explain certain phenomena concerning clay which are not well understood at present. But little work has been done on this line. Other problems are awaiting the joint attack of both sciences.

GLENNON GILBOY, Sc.D.:* The material embodied in Mr. Casagrande's paper constitutes, in the writer's opinion, one of the most significant developments of modern soil mechanics. From a scientific standpoint, the realization that the properties of a clay may, depending upon its structure, vary within wide limits has necessitated the development of new methods of investigation and the pursuit of research along paths hitherto unexplored. On the practical side, many existing ideas concerning the utility of pile foundations must now be considered doubtful, to say the least, and new lines of thought with regard to adequate foundation design are revealed.

Mr. Casagrande has pointed out that, in dealing with foundations on clay, the most satisfactory results will be obtained if the clay is allowed to remain, as far as possible, in an undisturbed condition. The writer feels that this principle is worthy of special emphasis. No engineer would think of improving the quality of the rock foundation under a dam by drilling holes in the rock and blasting it to pieces; nor of increasing the strength of a plate girder bridge by the use of rivets so large and so closely spaced as to reduce the strength of the steel itself to a small fraction of its normal value. Yet the destructive action of large groups of closely spaced piles driven into consolidated clay deposits is closely analogous; not quite so harmful, probably, but nevertheless similar. It is particularly interesting to note that if the structure of a consolidated clay is thoroughly broken down by such a group of piles, the properties of the material are so fundamentally altered that the entire deposit would settle appreciably *under its own weight*, without any additional building load, before equilibrium could again be established.

The writer has frequently been asked why all uncertainties in regard to the bearing value of a pile foundation could not be readily eliminated by testing the carrying capacity of individual piles. More specifically, suppose that all the piles intended to support a given building have been driven; that loading tests performed on several piles scattered throughout the group have shown that any one of the piles will not settle more than one quarter of an inch under a load of 10 tons; and that at least 40 tons of load must be applied to produce failure of any one pile. Is it not, then, reasonable to assume that, if the building load is so distributed as to produce a load of not more than 10 tons on any one pile, the building should not settle more than one quarter of an inch?

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* Assistant Professor of Soil Mechanics, Massachusetts Institute of Technology.

The answer is, of course, in the negative. A full discussion of the matter would be rather lengthy; therefore recourse may be had to a structural analogy. In Fig. A is shown a column of a building, with floor beams attached. Assume that the theory of structures is still in its infancy, so that analytical methods for determining the safety of the column are not available. It is decided, therefore, to load a single floor beam, such as *A*, with the full load it will be expected to carry.

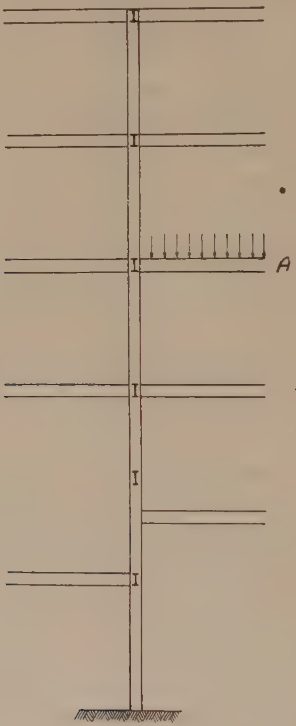

FIG. A

Suppose it is found that beam *A* carries its full load in a satisfactory manner. Can one assume, then, that the whole structure will be stable when fully loaded? Certainly not. The loading test on beam *A*, or a similar test on any other beam, proves that the beam is strong enough, and that the connection at the end is strong enough, to transfer the load to the column. But there is no guarantee that the column will not buckle when all the beams are loaded at once.

Thus it can readily be seen that a safe-load test on a pile merely proves that the pile itself will not crush, and that the "connection" between pile and soil is strong enough to transmit the load without a great deal of movement. But when a large number of such loads are transmitted to the soil, the soil itself tends to deform. Concerning the amount and rate of this deformation, which is the all-important item with regard to settlement, the loading test gives no information whatsoever.

In view of the foregoing considerations, Mr. Casagrande's contention regarding the detrimental effects of pile-driving in clay appears to be justified. It is quite possible that each individual pile will develop a satisfactory amount of resistance. But when all the piles are loaded, the remolded clay in which they are embedded is subjected to relatively heavy stress. This stress will produce a certain deformation; and, other things being equal, the deformation will exceed by far that which would have occurred if the clay had been left undisturbed.

For the sake of clarity, Mr. Casagrande has presented his material in the simplest possible terms. It should be noted, however, that the problems involved are actually of a rather complicated nature. While the fundamentals are fairly well understood, qualitatively at least, there are a great many points which must be thoroughly investigated before settlement analyses may be made with any degree of assurance. As a matter of principle, the writer agrees that pile foundations in clay should be avoided; but, as a matter of economy, such foundations may often deserve consideration, at least as a possible alternative. For this reason it is necessary to have some definite information on the amount and the extent of the disturbance created by the piles; on the distribution of skin friction and point resistance between pile and soil, both at the outset and during the consolidation process; on the relations between the soil stresses produced by a pile foundation and those produced by a mat or by caissons; on the proper size, length and spacing of piles, to produce the most favorable stress distribution and to minimize the undesirable effects of disturbance; on the possibility of drainage of water through wood or concrete piles, thus increasing the rapidity of settlement; and on the feasibility and economy of using something other than a driven pile — for example, open-end pipes jacked down, blown out, and later filled with concrete — to obtain the advantage of a pile foundation with a minimum of disturbance to the clay.

Investigations of the above questions and of other related items involve research programs of rather large magnitude. Small-scale tests might be used to some extent, but conclusive data could be obtained only from studies on full-sized operations. Plans have already been developed whereby the necessary information could readily be obtained. Their execution is impeded only by the lack of funds. It is hoped that this unfortunate condition will not prevail indefinitely.

H. A. MOHR:* Mr. Casagrande has presented one of the most understandable engineering papers it has ever been my pleasure to hear. Perhaps this is because he apparently has found a reasonable answer to several phenomena that have for a long time been taken for granted, as we accept taxes and debt.

It is fortunate for the profession that he has been so bold as to present the results for discussion. Now that he has performed his part, it is the obligation of the men with practical experience to recall experiences that will tend to prove or disprove the conclusions he has arrived at.

Fortunately, in our work — the making of borings for foundation purposes, installing caissons and driving piles — we have an unusual opportunity for observation relative to the subject of his paper.

For a number of years we have known that the boring samples of clay are always softer than the same material when uncovered in our caisson work. This is particularly so with soft and medium clay. When a clay is easily classified as soft, medium or hard this difference is not brought to one's attention; but when the sample is on the border between one or the other this difference comes to mind, and, generally, the classification of the next harder grade is designated, unless previous experience in that immediate vicinity has proven otherwise.

While we have known of this condition for years and have accepted it as a fact, and have been governed accordingly, we have had no direct proof as to why, nor have we made any attempt to determine the extent of the difference. The author has given an explanation of why, and an idea of its extent.

During the past three years we have had occasion to place the foundations for what is probably the heaviest building per square foot of plan area in Greater Boston; this work was done in two sections. The first half is on caissons carried to bedrock by the use of compressed air. The difficulties encountered in sinking these caissons, and the expense involved, caused the selection of a precast concrete pile type of foundation to be used on the last half. The sinking of the caissons afforded an excellent opportunity to study the ground conditions first hand. This information was very useful in planning the pile work so that trouble would be avoided and to insure a good job without experimenting. A general outline of the ground conditions is necessary for an understanding of the methods pursued. From street grade +17.0 to 0.0 fill; to −15.0 silt; to −22.0 hard yellow clay; to −30.0 medium yellow clay; to −80.0 soft clay; to −85.0 coarse gravel; to −95.0± hardpan; bedrock lay directly under the hardpan. The top 12 inches or so of the hard yellow clay was so dry that it would crumble when pressed between the fingers. The soft clay was very plastic, and under its natural pressure gripped the caissons to such an extent that they could not be moved by weights. Air pressure was of no use, as it would flow through the coarse gravel stratum and be lost. When the gravel was uncovered the water would rise and be constant at +11.0, one foot above mean high tide. The general excavation was carried to −17.0 to provide space for concrete water storage tanks. Pile cut-off was generally at −22.0±.

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* Gow Company, Inc., and Raymond Concrete Pile Company.

It was desirable to place the piles in such a manner as to cause as little disturbance as possible to the body of clay. The tanks were to be carried directly on it, and any possibility of damage to adjoining property, sewers and public utility conduits was to be avoided. The piles were 20 inches square and were to obtain their bearing resistance in the hardpan stratum. We knew that driving piles of this size and length in clay would cause a tremendous heave and displacement, which meant that some means must be adopted to relieve this condition. It was decided to excavate a hole with an auger for each pile. This hole was to be dug as deep as possible, but not so deep as to release the water in the gravel. The proper size of the hole was to be determined as the job progressed. The piles were on 3-foot centers. With these close centers we decided to bore staggered holes, and after driving the piles in these holes, to bore the intermediate holes.

The work was started on this basis, and no difficulty was encountered in the boring work. The auger would be withdrawn with a good volume of clay which could be cut with hoes and handled easily. But this condition changed radically after the first piles were driven. Apparently the pressures generated in driving the piles, and the displacement of the material caused it to change completely in character. We found it not only extremely difficult to dig the intermediate holes, but it was almost impossible to remove the material from the auger. For the balance of the work every hole was dug in consecutive order, and we spent the extra time being careful to prevent the auger from running into a hole already dug rather than contend with the changed condition caused by the first method of operation.

Further than that, we found it possible, after part of the piles had been driven, to continue to dig in holes for a considerable time, taking out material but gaining no depth. In fact, it was necessary to do this occasionally to relieve the pressure that developed. This pressure was of such an intensity as to raise a temporary construction trestle supporting a heavy steel pile-driver, and to heave piles already driven to refusal as much as five feet. Some of these piles had to be redriven as many as five times to make them stay in the original driven position.

This method of operation proved very satisfactory in accomplishing the desired results. There was some heave of the general area despite all of the precautions taken. At the completion of the work it was the opinion of all connected with it that the volume of clay removed practically equaled the volume of the piles driven into it. There is never any practical way of proving this on a construction job.

The experience on this work tends to prove the author's contention that the character of the clay is entirely changed by driving piles into it.

There seems to be no way of proving to what distance from the piles this disturbance is active. Every one acquainted with the heave caused by driving piles into clay knows that the disturbance is rather extensive. We have had instances where the heave amounted to as much as six feet. If the formation of clay is truly represented by the delicate structure outlined by the author it would seem that any bodily movement would destroy its equilibrium. Certainly one would be led to the conclusion that where the heave is extensive the entire area is so affected as to be classified as remolded clay.

This is a very interesting subject, and should command the attention of every engineer, architect and owner who has to do with underground work.

ARTHUR CASAGRANDE: I frankly admit that I have presented my paper with a great deal of uneasiness, realizing how radically different my conclusions are from established engineering practice. Therefore, it gave me a sincere pleasure to hear and to read these discussions by a number of experienced engineers and scientists. Each of these discussions represents a most interesting and valuable contribution to the subject, illuminating some points which I have not brought out clearly, or supplementing the material contained in the paper by important observations and pertinent remarks.

The following notes represent answers to questions which were brought up by Mr. Marston in his discussion.

So far, I have not found any clay with a very small or no difference in compressibility between the undisturbed state and the remolded state. The smallest differences I have measured were found in an extremely soft organic clay (muck) from the bottom of the Potomac River, Virginia, in connection with a study of hydraulic fills for the Mount Vernon Memorial Highway. It is very likely that for unconsolidated clays the compressibility in the undisturbed state increases with increasing rate of load application, due to a partial breakdown of the structure. In such a case the consolidation test on the undisturbed sample would have to be carried out with special precautions, if it is intended as a measure for the difference in compressibility between the undisturbed state and the remolded state.

We tried to make consolidation tests on Laurentian clays in horizontal and vertical directions. However, we did not succeed in cutting out vertical samples because they would always break along planes of stratification.

The percentage of scale-like particles was not determined in the Laurentian clays.

The compressibility and elastic rebound in the remolded state are doubtlessly controlled by the percentage of scale-like particles. However, in the undisturbed state they are dependent, also, on the structure of the clay.

I believe that fine-grained soils, consisting largely of bulky grains, may also exhibit structural effects similar to that of clay, if deposited under conditions which cause flocculation of the grains during the sedimentation process. However, I have had no opportunity to examine such soils in the undisturbed state.

The term "structure," as applied to cohesionless soils, has, of course, a different meaning from that used in connection with the properties of clays. In the former it refers to the density of the deposit. An excellent example of structural differences of this kind in sand was offered by Mr. Marston in his discussion.

Pile-driving or other manipulations on the ground tend to increase the density of a sand, and to decrease its compressibility, while in clay such actions tend to destroy the natural structure, and therefore to increase its compressibility.

VISUAL INSPECTION OF METAL ARC WELDS

Frank B. Walker,* Member, Boston Society of Civil Engineers

Although the greater percentage of metal arc welds are good, certain characteristics of arc welding which govern the quality of the work are emphasized in the following paper, in order that good welds may be readily distinguished from bad welds by visual inspection.

There are three important things which cause variations in the appearance of welds. From these differences in the appearance of the welds or base (parent) metal, the quality of the weld can be determined within reasonable limits. These three things are —

Length of Arc.
Temperature of Arc.
Cleanliness of the Base Metal Surfaces.

Length of Arc

The steel forming the weld passes from the negative electrode rod through an envelope of gas formed by the arc, to the positive base metal.

The distance from the end of the electrode to the base metal is called the length of arc and is normally about one-eighth of an inch. It is known that the quality and strength of the weld will be greater with a short arc and less with a long arc, other conditions being equal.

If the arc is too long, the gaseous envelope is too large. This spreads the heat and steel over too much space, and gives a porous weld of shallow penetration with wide overlap (incomplete fusion at the weld edges). It also spatters a pattern of small drops of molten steel over a considerable area, frequently several inches away from the weld.

A short arc is steady, and, being more easily controlled, spatters little or no drops, and produces a sound weld of deep penetration with no overlap.

If steel drops are spattered more or less thickly near a weld, or the weld shows overlap, it is probable that the weld has a shallow penetration, is porous, is not ductile, and is of too low strength. A few scattered drops near a good weld may be due to the welder purposely striking or breaking the arc, but a constantly held short arc will not spatter drops.

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* Chief Engineer, Eastern Massachusetts Street Railway Company, Boston, Mass.

Temperature of Arc

The quality of a weld is largely dependent upon the heat of the arc, which depends on the amperes used, more being required for larger than smaller rods. Laboratory tests show that good welding can be done only within a very narrow temperature range.

If the arc is too cold, the resulting weld resembles cold solder, with incomplete fusion, — that is, it inadequately unites the edges or root of bead with the base metal, and has very little strength. In appearance the weld is uneven, has thick and thin places, possibly with drops of steel adhering to the bead, or the edges of the weld curl under instead of flattening out and merging into the base metal.

If the arc is too hot, the weld steel is burned, or partially so, and the welds are hard, brittle, porous, and probably have incipient cracks, if not actual ones. Such welds will in a short time (within twenty minutes, if very hot) show their burnt condition by turning a rusty red, or by the distinct bluish tinge of cinder-like burned steel.

A weld made at the proper temperature will remain clean and will have a bright, metallic luster for a long time. A weld, being a cast steel, will not corrode very quickly if not burnt.

If a rusty red or cinder appearance develops soon after welding, the steel has been burned and is of low strength and is unreliable.

All welding machines have a rheostat to regulate the current at the arc. The rheostat should be manually varied with changes in line voltage or size of electrodes, to give the correct welding heat.

Welding Surfaces

Good welding cannot be done through a coating of paint, scale, rust, dirt, oil or anything else, except linseed oil. The actual surfaces under the weld cannot be seen after the welding is done, but the inspector can tell by the appearance of near-by surfaces whether they have been cleaned off with steel brush or cold chisel to the bare metal, or at least to a thin, tight mill scale before welding was started. It is of the utmost importance to have clean surfaces.

Position and Size of Weld Bead

Welds should be of the right size, in the proper place, and unite the edges of the base metal properly. The weld joining the edges of two plates in a butt weld should more than fill the V, or opening, and where uniting a plate and angle it should be full and uniform as to location, size, etc. (Fig. 1.)

It is common practice for the welder to swing his arc back and forth across the weld as he progresses along the weld seam, making a "half moon bead," which operation unites the edges of the base metal and the bead and at the same time makes the weld of the proper thickness.

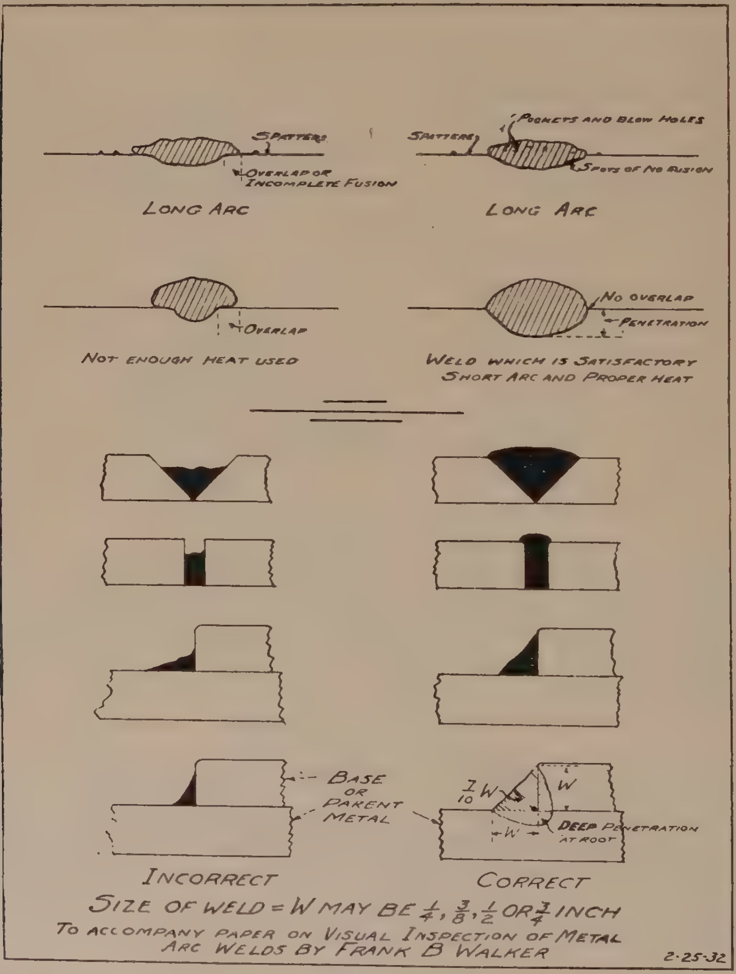
SPATTERS
OVERLAP OR INCOMPLETE FUSION
LONG ARC
POCKETS AND BLOW HOLES
SPATTERS
SPOTS OF NO FUSION
LONG ARC
OVERLAP
NOT ENOUGH HEAT USED
NO OVERLAP
PENETRATION
WELD WHICH IS SATISFACTORY
SHORT ARC AND PROPER HEAT
BASE OR PARENT METAL
$\frac{7}{10}$W
W
W
DEEP PENETRATION AT ROOT
INCORRECT
CORRECT
SIZE OF WELD = W MAY BE $\frac{1}{4}$, $\frac{3}{8}$, $\frac{1}{2}$ OR $\frac{3}{4}$ INCH
TO ACCOMPANY PAPER ON VISUAL INSPECTION OF METAL ARC WELDS BY FRANK B WALKER
2-25-32

FIG. 1

An arc weld is sometimes referred to as a flat, vertical or an overhead seam. A simple way to distinguish between them is to consider any seam laid down on a horizontal or nearly horizontal plane as flat, those welded on vertical or nearly vertical plane as vertical, and all others as overhead. Flat seams require less skill on the part of the welder than the vertical or overhead seams.

GENERAL

Welds or base metal should not be painted until after inspection and approval.

Each welder should stamp his own number near his welds for identification purposes.

When coated welding rods have been used, it is desirable, when making an inspection, to clean off the weld and base metal with a wire brush, as there will be an accumulation of oxidized matter, slag or impurities.

It is assumed that the base metal is suitable for metallic arc welding, and that the welding rod used is properly treated during manufacture (but not necessarily coated), is uniform in quality and is weldable. If the rod does not conform to these characteristics, the resulting weld will have some of the faults previously described. The use of a rod which is not uniform will usually make a weld not uniform.

A run of poor welds may sometimes be found in a good welder's work. This may be due to a variety of reasons, such as the use of a welding rod of poor quality, rapid changes of line voltage not followed quickly by rheostat adjustments, poor electrical ground return, or the welder permitting his helper to practice when no one is looking, or carelessness.

When welds are tested, either in the field or laboratory, it is suggested that the tests be made in the presence of the welder, as it is good discipline for the welder, especially when poor work is found, and it is always instructive for both welder and inspector.

A million and a half lineal feet of arc weld seams have been made during the past thirteen years by at least one hundred and fifty different welders under the supervision of the author, and all the bad welding which was found on this work can be definitely attributed to the following three causes only:

1. Arc too long.
2. Arc too hot or cold.
3. Surfaces not clean.

APPEARANCE OF A GOOD WELD

A good weld —
Has no drops spattered near by.
Has no overlap (incomplete fusion at weld edges).
Has both edges merging smoothly into base metal.
Has uniform bright metallic luster.
Has uniform size and thickness.
Has evidence that base metal was clean before welded.

NOTE

The specimens showing good and bad welding on exhibition in the Society rooms were made by the New England Structural Steel Company and the Leake & Nelson Company to illustrate the points discussed in this paper.

Any one interested in structural welding should read the recently published report of the Committee on Structural Welding of the American Welding Society. It contains data on qualifying welders and allowable unit stresses for different sizes of welds with factors of safety.

Comments by Past President Harry E. Sawtell

This paper by Mr. Frank B. Walker, Chief Engineer of the Eastern Massachusetts Street Railway Company, on the subject of "Visual Inspection of Metal Arc Welds," is a valuable one.

Having discovered that Mr. Walker has had long years of experience in the practical work as well as the theory of metal arc welds, I confess to a feeling of pleasure at his consenting to put his notes together in the form of a paper.

It is a timely subject and a paper which will help many men in their work, whether it is making or inspecting arc welds.

The committee that has compiled subjects for research included this item in its published list; therefore this is one of the first papers to cause a cancellation to be made of any subject in the list.

The specimens of good and bad welds which Mr. Walker is placing in the Society rooms for exhibition will be very helpful when examining them in connection with this paper.

Discussions of this paper will be very welcome.

NEWLY ELECTED HONORARY MEMBERS, BOSTON SOCIETY OF CIVIL ENGINEERS

HONORARY membership in the Boston Society of Civil Engineers has been conferred on the following during the year ending March 16, 1932:

Prof. C. Frank Allen, senior member of the Society, a member since March 24, 1875, elected Honorary Member March 16, 1932.

Mr. Charles T. Main, a member since June 17, 1891, elected Honorary Member February 17, 1932.

Dr. Karl T. Compton, President of Massachusetts Institute of Technology, elected Honorary Member January 27, 1932.

The present list of honorary members is as follows:

John R. Freeman.
Prof. C. Frank Allen.
Charles T. Main.
Dr. Karl T. Compton.

At the annual meeting of the Society, held on March 16, 1932, diplomas of honorary membership were presented to Mr. Charles T. Main and Dr. Karl T. Compton by Mr. Harry E. Sawtell, President. Prof. C. Frank Allen, who is traveling, was unable to be present at this meeting. Prior to the presentation of these diplomas, a biographical sketch of Mr. Main was read by Mr. Irving E. Moultrop, member, and a biographical sketch of Dr. Compton was read by Past President Robert Spurr Weston.

Charles T. Main

Charles T. Main is a native of Marblehead, Massachusetts, where he was born February 16, 1856, the son of Thomas and Cordelia Reed Main. The elder Mr. Main was a native of Marblehead, and his mother came from Plymouth. Both were members of old American families, established in colonial days.

Mr. Main graduated from Massachusetts Institute of Technology in 1876, with the degree of B.S. from the department of mechanical engineering, and remained at the Institute for three years as an assistant in that department.

After serving for a little over a year as a draughtsman at the Manchester Mills, he became engineer of the Lower Pacific Mills, Lawrence, Massachusetts. While remaining in control of all engineering work, he was promoted to be assistant superintendent in 1886 and to the superintendency of the mill in 1887. During the period from 1881 to 1892 the plant was largely reorganized and rebuilt.

Mr. Main's inclinations led him so strongly to engineering that he left the Pacific Mills, and, after devoting about a year in miscellaneous engineering work in Providence, he formed a partnership in 1893 with Mr. F. W. Dean for conducting an engineering business largely in industrial plants.

In 1907 this partnership was dissolved and Mr. Main conducted his business under his own name until July, 1926, when it was changed to Chas. T. Main, Inc.

From 1893 to the present time a large number of industrial plants have been designed and constructed and other plants reorganized under the direction of Mr. Main. This involved the engineering for industrial plants developing into steam power plant work, and water power into hydro-electric developments. Among the largest undertakings in the group of industrial plants are the Wood Worsted and Ayer Mills in Lawrence, and, in the hydro-electric field, developments for the Montana Power Company, aggregating about 300,000 horsepower.

Mr. Main has served as consulting engineer on many projects and as expert witness, or referee, in important legal actions, and also has placed valuations on numerous industrial properties. He has written numbers of papers on engineering subjects.

His interest has been extended to public affairs and in this connection he has held several municipal offices in order to do his share of public work, and for the purpose of advancing good government. He was an alderman of Lawrence for three years, and also a member of the board of trustees of the public library, and member of the school committee in that city. For eleven years he was a member of the water and sewer board at Winchester.

Mr. Main is a member of the American Society of Mechanical Engineers, and was a manager of the organization for three years, and president in 1918. He also is a member of the American Society of Civil Engineers. He was president of the Boston Society of Civil Engineers in 1911 and president of the American Institute of Consulting Engineers in 1929.

For several years he has been a life member of the Corporation of Massachusetts Institute of Technology, and a member of its executive committee. Mr. Main is a Fellow of the American Association for the Advancement of Science and a Fellow of the American Academy of Political and Social Science.

At the request of France Mr. Main was a member of the American Engineering Commission which visited France immediately after the World War regarding rehabilitation problems.

Some of Mr. Main's other activities and associations are as follows: life member of the University Club, member of the Engineers Club of New York, member of the Boston Chamber of Commerce, member of the Exchange Club of Boston, member of the Winchester Country Club, past president of the Engineers Club of Boston and now an honorary member of that club, and trustee of the Winchester Savings Bank.

Mr. Main was married in 1883 to Miss Elizabeth F. Appleton. They have three children, Charles R., Alice A., and Theodore Main.

Response by Mr. Charles T. Main on Receiving Diploma of Honorary Membership

Mr. President and Fellow Members: The President and Mr. Moultrop, who have preceded me, have said everything I intended to say. As the President has said, I have been a member of this society for more than forty years. During that period I have served on many committees, on the Board of Government, and as President of the Society. I consider, as the President has said, honorary membership to be the highest distinction that this Society can confer upon anybody.

I sincerely thank you for the honor which you have conferred upon me. I hope that by elevating me to this position you have not "put me on the shelf," as I should like to be of further service to the Society in advancing its interests at any time when I can be of assistance.

Dr. Karl T. Compton, President of the Massachusetts Institute of Technology

Dr. Karl Taylor Compton, eleventh president of the Massachusetts Institute of Technology, was born at Wooster, Ohio, September 14, 1887. His father, Dr. Elias Compton, a Presbyterian clergyman, and emeritus dean and professor of philosophy of Wooster College, is the head of a notable family, which includes another scientist, a brother of President Compton, Dr. Arthur H. Compton, professor of physics at the University of Chicago, who was awarded the Nobel Prize Laureate in Physics in 1927.

Dr. Compton got his bachelor's degree from Wooster College in 1908, his master of science degree in 1909, and his doctor's degree at Princeton in 1912. In 1923 Wooster College gave him the honorary degree of doctor of science, as did Lehigh University in 1927 and Princeton in 1930. In the same year Brooklyn Polytechnic Institute gave Dr. Compton the honorary degree of doctor of engineering, and Harvard, the degree of doctor of laws.

Dr. Compton was instructor in chemistry at Wooster College from 1909–10, and instructor in physics in Reed College at Portland, Oregon, from 1912 to 1915. Returning to Princeton in 1919 as assistant professor of physics, he was later promoted to the rank of professor of physics, and afterwards research professor and chairman of the department of physics at Princeton, a position he occupied until just before he was inaugurated as president of the Massachusetts Institute of Technology on June 6, 1930.

He is the author of many publications dealing with researches in photo-electricity, ionization of gases, soft X-rays, spectroscopy in the extreme ultra-violet field, fluorescence and dissociation of gases, electric arcs and other types of gas discharge, thermionic and other phenomena.

He is a Fellow of the American Physical Society and of the American Optical Society; a member of the National Academy of Sciences, the American Philosophical Society, the American Chemical Society, the American Electrochemical Society, the Franklin Institute, the Deutscher Physikalischer Gesellschaft, the American Association for the Advancement of Science, the American Institute of Electrical Engineers, the American Academy of Arts and Sciences, the Society for the Promotion of Engineering Education, and also of the Alpha Tau Omega, Phi Beta Kappa, and Sigma Xi fraternities.

Dr. Compton is a past president of the American Physical Society, 1927–29, and a past chairman of the division of physics of the National Academy of Sciences, 1927–30. He is also a member of the executive committee of the National Research Council and of the American Association for the Advancement of Science, and chairman of the physics subcommittee of the National Research Council on the Chicago, 1933, Exposition. He is consulting physicist for the General Electric Company and the United States Department of Agriculture, and a member of the advisory board of Bartol Research Foundation. During the World War he was aeronautical engineer in the Signal Corps, and later an officer of the Research Information Service. In 1918 he was associate scientific attaché to the American Embassy in Paris.

Dr. Compton married a daughter of J. Corrin Hutchinson, emeritus professor of Greek in the University of Minnesota. They have three children.

Response by Dr. Karl T. Compton on Receiving Diploma of Honorary Membership

Mr. President, Mr. Weston and Members of the Boston Society of Civil Engineers: I scarcely know how to express my appreciation and gratitude for this very high honor which has been bestowed upon me, an honor which I feel is all the higher because it is bestowed at the same time upon my great friend and colleague, Mr. Main.

I think it is true that any man, whether he is a rogue or only a common person, likes to be found in respectable company. The words "old and respectable" go together just as "crime and shame" do, and I think there is really a logical connection between these two words. However, I will not elaborate upon this.

It is certainly true that the civil engineering profession is the oldest of the engineering professions. In Man's moving about on the surface of the earth and endeavoring to make a suitable place to live in, from the earliest times he has practiced civil engineering. So I feel in being thus honored by election to a civil engineering society I am joining one of the oldest, and therefore one of the most respectable, professions.

At the same time, I understand that the Boston Society of Civil Engineers is the oldest civil engineering society in this country, and if it is not the mother it is at least the foster-mother of the other local societies and of the American Society of Civil Engineers.

So therefore in this again I feel that I am being honored by election to membership in the Boston Society of Civil Engineers, in that I am thus becoming a member of the oldest, and therefore the most respectable, of the civil engineering societies.

I have been trying to find out what might have been the excuse of the committee for conferring this honor upon me. I have thought of three possibilities and have surveyed them one after the other, but I am sorry to say that I have found each of these possibilities entirely inadequate.

These three have to do with some items of my biography not disclosed by Mr. Weston in his very comprehensive survey. My first thought was that it might have been a matter of heredity. Mr. Weston, in mentioning my father, did not disclose the fact that he worked his way through college by serving as a civil engineer, and that as a boy I used to hear many stories of his experiences as a surveyor.

The second possibility, perhaps, was the fact that during my first year of postgraduate work I happened to be student assistant in surveying in the College of Wooster, and I had charge of the preliminary survey of the first mile of model road built by the State of Ohio; but this does not appear in "Who's Who," and I am sure this was not an adequate reason.

My final experience as a civil engineer was during a short term at Reed College in Portland, Oregon. I was a member of the Committee on Athletics, and it was put up to me to try to help the Barber Asphalt Company complete a contract made with the college, to build a series of tennis courts. These were to be built with a slope of 6 inches from the back to the center, and were to be perfectly smooth, to prevent pooling. After a rain the water was supposed to drain away promptly. This was very necessary in a climate like that of Portland.

The courts had been built and rebuilt about three times, but pools still formed on them after rains, and as a result the Barber Asphalt Company had not been able to collect their money. So I was sent out with a level and a rodman, with instructions to drive spikes on the asphalt of the tennis court, each adjusted exactly to the right level, so that when the large-footed negroes who laid the asphalt came to spread it they could get it very accurately to the levels of the spikes and consequently the problem would be solved.

I was not a very experienced engineer. I went out on the courts and set up my level on a corner of the asphalt and got the level and proceeded to make the survey, but happening to glance at the instrument I found that the level had tilted somewhat. Then I realized that the asphalt expanded under the action of the sun, and also contracted every time the sun went behind a cloud, and this action was affecting the level. Finally, however, by getting it on Mother Earth I succeeded in placing the spikes.

But when the workmen came to lay the tennis courts it was discovered that they were not used to laying asphalt level with the top of tenpenny spikes. They simply kicked them out of the way and laid the asphalt without them. And they did a very excellent job.

The next time the sun went under the clouds and came out again there were holes, and I do not know whether the Barber Asphalt Company has ever collected their money.

That is the sum total of my qualifications professionally for this honor. But I realize that in conferring this honor you have had two thoughts in mind: one is a kindly expression of "God speed" to me in my connection with the Institute, in which you as well as I have an interest; and the other is your very kind expression of good will to the Institute itself.

I was asked by the President to speak for not more than ten minutes. I think, perhaps, there is just one more thought I might leave, of a serious type.

As I have been thinking, the last couple of years, particularly of the various engineering professions, one thought has been very striking to me, and it becomes more so the more I think about it; that is, the very short span in human history within which the engineering developments of the type which we now think of as the modern activities of engineering have actually been practiced in the present manner.

The art of transportation is an example. Modern transportation, except by foot and by horseback, extends back not over one hundred years. The latest developments are within a much shorter time than that; the airplane and the automobile within less than a quarter of a century; and the dirigible still on the horizon. This represents a period almost infinitesimal in proportion to Man's span of life.

In the art of communication, again. It is literally true that the most rapid methods up to fifty years or more ago have been the old-fashioned method used by the Egyptians and by Ghengis Khan, namely, postriders, with their relays. And only within a quarter of a century, or less, has it been possible to talk by electric communication to the most distant parts of the earth, with a time lag no greater than that with which my voice reaches the back of this room. So that, again, is very recent.

Then there are the ways of getting from the earth those materials on which our life depends, such as mineral resources and agricultural resources. Developments in those have been very, very recent, and yet in comparison with these, if we consider the work of the civil engineer, we find it going back to antiquity and extending down to the present time with that "hoary age" and consequent respectability mentioned before.

Another point connected with that thought is that the work of the civil engineer goes back into antiquity farther than does that of any other group of applied science, and perhaps covers a greater range of activities than does any other branch of engineering. Nevertheless, what to you at the present time is the most interesting is the *future* of civil engineering and the fact that it is making just as rapid developments as any of the other activities just mentioned. This makes the work of the civil engineer particularly interesting. It is of interest both because of its historical background and also in its future development, on account of the research which is going on at the present time.

In conclusion, I wish again to express my very great appreciation of this distinguished honor.

INTERNATIONAL FACTORS IN THE DEPRESSION

By Prof. Ralph Evans Freeman *

(Presented at the Annual Meeting of the Boston Society of Civil Engineers, March 16, 1932)

There is no agreement among economists as to the part played in our present industrial tragedy by international disturbances. Some economists believe that our troubles are almost entirely of foreign origin, while others assert that external factors have had very little to do with our unhappy condition. The truth probably lies between these extremes. While the fundamental causes of depression are to be found in the nature and functioning of our productive institutions and in the psychology of the human beings who use them, the explanation of its extraordinary depth and duration must be sought in the chaotic state of international relations. When the depression arrived the nations were not in equilibrium in their relations with each other, and, like a fighter who is caught off balance, the economic world was unprepared to receive the blow which descended.

A nation is in equilibrium with the rest of the world when its in-payments are equal to its out-payments. These payments originate from a great variety of transactions, — imports and exports of merchandise, borrowing and lending, interest payments, reparations, foreign travel, immigrants' remittances and many others. Some of them give rise to claims against the inhabitants or government of the country; others give rise to claims against foreigners. When these claims and counter-claims are equal, the country is internationally stable; and if it happens that all the others are in a similar position, it may be said that the industrial system as a whole is in a state of international equilibrium.

Suppose that, under these hypothetical conditions of perfect adjustment, a nation increases its foreign lending. Out-payments exceed in-payments and the balance is upset. Almost immediately forces are set to work which tend to restore the balance. The expansion of out-payments causes a contraction of bank deposits within the country producing a drop in the level of internal prices. At the same time, because of an addition to the purchasing power of foreign countries, prices abroad move upward. With domestic prices falling and foreign prices rising it becomes

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* Associate Professor of Economics, Massachusetts Institute of Technology, Cambridge, Mass.

profitable to export more goods and services. As exports grow and in-payments multiply, income and outgo tend to be equalized.

These stabilizing adjustments in price levels may be facilitated by gold movements. When an increase of foreign lending causes out-payments to exceed in-payments, the rate of exchange of the money of the lending country falls. In this event foreign borrowers may find it advantageous to take payment in gold rather than in bills of exchange payable in the currency of the lender. As gold is withdrawn, bank reserves in the creditor country are diminished, and this may lead to a contraction of bank credit which will depress the price level. But the arrival of gold abroad will have an opposite and inflationary effect on foreign price levels. Exports will therefore increase or imports diminish until equality of in-payments and out-payments is restored.

Whenever the equilibrium is disturbed, a subsequent change of relative price levels, by giving rise to an appropriate flow of goods, tends to stabilize the situation. Perhaps the original source of unsettlement is in merchandise trade; perhaps in the volume of interest payments; or in the amount of foreign travel. It does not matter what the source of disturbance, price changes at home or abroad, or both, will start a flow of goods to restore the balance. If in-payments exceed out-payments, the flow will be inward, induced by a fall of prices abroad and a rise of prices at home. Goods move according to the demands of the situation.

Before the war this machinery of adjustment worked with a fair degree of smoothness and efficiency. Though derangements were constantly appearing, alterations in national price systems, by affecting imports and exports, were constantly at work to remove the discrepancies. The large out-payments which the United States as a debtor had to make for interest on her outstanding obligations were offset by in-payments from a surplus of exports over imports. The enormous in-payments which Great Britain as a creditor received from her foreign investments were counterbalanced by out-payments from a surplus of imports over exports. Canada, with a large income from current borrowings, was kept in equilibrium in the same way. And so, in regard to most of the other countries, the changes which occurred to disorganize their economic relationships with the rest of the world were never so formidable that the corrective machinery could not cope with them.

But not so in these later years. Something has gone wrong with the stabilizing mechanism. When nations get out of balance in their international dealings, the normal forces of adjustment do not seem to be able to bring them back. This is the explanation for the chronic instability of international economic relationships that now exists. It reacts unfavorably upon every country that sells in foreign markets or has to rely upon foreign sources of supply, and must, as I have said, be held largely responsible for the extraordinary length and severity of the present depression.

For this condition I would suggest three principal causes: (1) the magnitude of post-war disturbances and the suddenness with which they have occurred; (2) the fact that national price levels have been prevented from undergoing the changes necessary to remove these disturbances; (3) international trade restrictions that have obstructed the flow of goods between nations.

1. The Violence of Post-War Disturbances

The effect of the sudden introduction of large inequalities in international payments is perhaps best illustrated by the case of Germany. The peace treaty brought reparations; and the exigencies of the reconstruction period after the war brought new foreign debts with their heavy burden of interest charges. To balance these enormous out-payments the German nation was faced with the necessity of developing an equal amount of in-payments. We know how she attempted to do this. She sold securities abroad, chiefly to the people of the United States, and used the proceeds to balance her international account. But, of course, this was only a temporary device. The additional obligations merely aggravated the situation by increasing the interest charges. It was obvious that, sooner or later, Germany would have to create the necessary volume of additional in-payments through a surplus of exports over imports.

This required a radical readjustment of Germany's commercial life. For many years she had been buying more goods from foreign nations than she had been selling to them, and her economic system had become adapted to such conditions. To reorganize this system fundamentally on the spur of the moment was not possible. No nation can do this, especially one which, like Germany, is dependent upon imports for a large part of the materials necessary to her manufacturing industries. Yet she did make some progress in this direction, and even achieved a slight export surplus in 1930. But more than a slight surplus was needed. In fact, never before had the machinery of international settlements been called upon to make such a tremendous adjustment as was required in this case. That it failed to do so is not surprising.

In 1928, to make matters worse, the temporary offsetting in-payments from American lending were suddenly diminished. The investors of the United States lost their enthusiasm for foreign securities. This left Germany still more out of gear with the rest of the world, and an even greater drag upon the economic progress of other nations. With enormous interest payments to make, and insufficient means of doing so, she became a serious menace to international stability. In 1931 her approaching insolvency undermined confidence in the world's credit structure, helped to force Britain off the gold standard, and brought other nations to the verge of financial breakdown. The repercussions were felt in this country in the shape of a further decline of security prices, bank failures and the hoarding of currency.

The international disequilibrium of Germany is matched by that of the United States. In our case the position is reversed. Our industrial life has become adapted to the export surplus which was required by our pre-war debtor position; but our new creditor status demands a surplus of imports. To balance in-payments from the receipt of interest, offsetting out-payments must be created. For a time we accomplished this by lending abroad. But the change in our investment policy which took place in 1928 removed that balancing item and we find ourselves seriously out of adjustment in our international account. To transform ourselves, in the span of a few years, from a net exporter to a net importer is just about as difficult for us to achieve as for Germany to effect a similar transformation in the opposite direction.

France has been a disturbing factor also; but in a different manner. During the last few years she has managed to accumulate a large amount of capital which has been invested in short term loans in London, New York and other foreign money markets. Such funds can easily be shifted; and the French, to serve their purposes both financial and political, have adopted the practice of transferring them frequently from one center to another. These transfers involve large out-payment from the countries from which the funds are withdrawn and large in-payments into the countries in which the funds are deposited, thereby unsettling the currency and banking systems of the former, and becoming a source of constant uneasiness to the latter. French financial policy, therefore, is responsible for many sudden changes in international payments, and has contributed to the defective operation of our stabilizing machinery

2. MALADJUSTMENT OF NATIONAL PRICE SYSTEMS

The second principal cause of trouble has been the failure of price levels to respond promptly to the demands of post-war conditions. The experience of Great Britain may be cited as an example. When she returned to the gold standard in 1925, with the pound at its pre-war gold value, a reduction of her prices and wages was required to counteract the increase in the value of her money. Because she failed to make this adjustment, her price system was out of line with the prices of other nations. Her production costs were so high that her export industries could not compete in foreign markets. Imports were attracted by her high prices; she found herself out of balance and losing gold. This state of affairs greatly contributed to her currency difficulties last year.

When national price levels get out of alignment it is difficult to be certain which are in line and which are not. Each country thinks that all the others are out of step but itself. The British, for instance, insist that prices in France and the United States were kept too low. As evidence of this they point to the ability of these countries to attract gold. Gold flows to those places where its value is highest, where the greatest number of other things can be obtained in exchange for it; that is, where the level of commodity prices is low. The British argue that if France and the United States had used all their gold as the basis of credit expansion and price inflation, they would have attracted goods instead of gold, and thus relieved the strain on the banking reserves of other nations.

A great deal of international price maladjustment is due to expansion of production during the war. Countries whose chief products are crude foodstuffs and raw materials have, in recent years, been adversely affected by the decline of the prices of their exports in relation to the prices of their manufactured imports. Stimulated by the extraordinary demand of the war period and by the use of improved equipment and methods, the production of wheat, rubber, cotton, sugar, coffee, hides, pulpwood and other foods and materials was greatly increased. But when the war stimulus disappeared, output was not sufficiently curtailed, and consumption did not grow fast enough to maintain prices. Australia is an outstanding example of a nation caught in the toils of overproduction. Her economic life was thrown into confusion by the fact that the prices of the goods she sold abroad declined disproportionately to the prices of the goods she bought. India, Canada, Egypt, British Malaya, Cuba, Hungary, the South American and Balkan countries were subjected to a similar experience.

3. International Trade Restrictions

The third great reason for the breakdown of our international economic machinery is to be found in the obstructions that have been raised to the flow of goods. To shut out foreign competition, governments have increased their customs duties and have thereby rendered still more difficult the task of apportioning to the different nations the imports and exports necessary to equalize their international payments. Since national price systems have to be adjusted to enable commodities to hurdle the tariff walls, and since the walls can be raised higher and higher, almost without limit, the task of adjustment is apparently hopeless. The prevailing idea seems to be that a nation may sell goods with profit to itself, but that imports are to be avoided like the plague. The inevitable outcome of this policy is to contract the volume of international trade and to defer indefinitely the attempt to restore the nations to a position of equilibrium.

But tariffs are not the only obstacles to be overcome. Of similar effect are the restrictions upon dealings in foreign exchange which many nations have imposed. In some cases, especially in central Europe and the Balkans, importers have to secure a permit from the government in order to obtain the foreign money which they need to carry on their business. The authorities are thus enabled to veto any imports that are likely to compete with domestic industries. They are able to direct the international flow of merchandise to serve political ends, to extend favors to friendly nations, and to injure the commerce of their enemies.

The bankers' committee under the chairmanship of Mr. A. H. Wiggin, which issued its report from Berlin in January of this year, summarizes the effect of international trade restrictions in the following sentences:

> The nations of the world are contending each for a disproportionate share of a *dwindling* world trade. With a different policy they could share with one another an *expanding* world trade. It is essential that trade policy should permit goods to move in the settlement of international debts, and that countries should make markets for one another. With trade lines open, labor now idle in one country could be at work producing goods to exchange for goods which would be produced by labor now idle in another country. The present extreme crisis must bring home to all peoples of the world the fact that all countries grow poor together. The obverse is as true. All countries grow rich together. A lightening of burdens and a greater freedom of trade enriching one country will enrich all.

The present condition of anarchy in international affairs surely emphasizes the fact that modern civilization is not constituted to withstand the shock of a world war. Improvements in transportation have drawn the nations close to each other, and national industrial specialization has made them interdependent. Economically the world has become a complex organism which cannot thrive when its parts are disconnected. Therefore, the disintegration, which takes place during a war, and which is continued by the intensification of nationalism following a war, cannot but do incalculable damage.

While economically united, we are politically divided. The engineer has pushed ahead of the politician. By enabling us to harness new sources of power, by placing at our disposal improved machinery, better equipment and more efficient productive processes, the engineer has created a unified economic world. But the politician has failed to supply the basis of political unity which the new technique demands. We are therefore compelled to impair the functioning of our economic system by breaking it up into national fragments. We are forced to pour the new industrial wine into old political bottles.

In fact, it is apparent that mankind is trying to live a double life, — a life on a national plane devoted to the conservation of national markets and national banking systems, and a life on an international plane devoted to the promotion of international financial and commercial intercourse. And it does not appear that we shall be able to live this double life successfully so long as we adhere to the ideal of national power and independence which has been the chief goal of modern political striving. Yet if we abandon this ideal we may have to sacrifice our racial integrity, self-government, national culture, and other elements of nationalism upon which we place the highest value. Can we continue to serve two masters? Can we retain the valuable elements of nationalism and at the same time reject those pernicious elements which threaten to paralyze our international trade?

The general opinion seems to be that the only way in which this can be accomplished is through greater co-operation between nations. In spite of the repeated failures of conventions, conferences, treaties and leagues we must continue to struggle for international co-operation. We must subordinate the selfish interests of belligerent nationalism to the welfare of society as a whole. Perhaps there is some reason to hope that the disorder which now delays the recovery of business will bring the nations to their senses.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

MARCH 16, 1932. — The eighty-fourth Annual Meeting of the Boston Society of Civil Engineers was held today at the Boston City Club, and was called to order by the President, Harry E. Sawtell, at 4.30 P.M.

The reading of the minutes of the previous meeting (February 17, 1932) was omitted, as these will be printed in the March JOURNAL. The minutes of other previous meetings during the year were approved as printed in the JOURNAL of various issues.

The President announced the death of Edwin H. Rogers, who died on March 6, 1932, and who had been a member since April 21, 1909; and of Granville Johnson, who died on March 12, 1932, and who had been a member since March 17, 1908.

The annual reports of the Board of Government, Treasurer, and Secretary were presented. Also the reports of the following committees: Library, Membership and Publicity, Registration and Licensing of Engineers, Social Activities, Subsoils, Research, Run-off, Welfare, and Relation of Sections to main Society.

Voted, That these reports be accepted and placed on file. The President stated that these will be published in the April JOURNAL.

Voted, That the incoming Board of Government be authorized to appoint such committees as it deems advisable.

The Secretary read the report of the tellers of election, William S. Keefe and William C. Hawley, and in accordance therewith the President declared that the following officers had been elected for the ensuing year:

President — Ralph W. Horne.
Vice-President (for two years) — John B. Babcock, 3d.
Secretary — Everett N. Hutchins.
Treasurer — Karl R. Kennison.
Directors (for two years) — John C. Moses, Edward Wright.
Members of Nominating Committee (for two years) — W. W. Bigelow, Erwin Harsch and William D. Henderson.

The President introduced Ralph W. Horne, the newly elected President, who responded briefly. The other officers elected were asked to rise as their names were read.

The retiring President, Harry E. Sawtell, then delivered his annual address on "Society Research."

Voted, That the Society express its appreciation to the retiring President for his success in administering the affairs of the Society during the past year.

The meeting adjourned to assemble again at 7.15 P.M., the annual dinner being held during the interim.

The President, Harry E. Sawtell, then called the meeting to order in the auditorium for the presentation of prizes.

He requested Prof. H. K. Barrows, member of the Desmond FitzGerald Award Committee, to present the Desmond Fitz-

Gerald medal to Mr. H. Kenneth Fairbanks, for his paper on "Wyman Dam and Hydro-Electric Power Development at Bingham, Maine," presented at the meeting of the Boston Society of Civil Engineers on November 18, 1931, and printed in the December, 1931, JOURNAL.

The President announced that the Designers Section prize of books, suitably inscribed, had been awarded to J. Stuart Crandall, for his paper on "Piles and Pile Foundations," presented at the meeting of the Designers Section on March 10, 1931, and printed in the May, 1931, JOURNAL. Mr. Charles R. Main, chairman of the Designers Section award committee, made the presentation to Mr. Crandall.

The President stated that the Board of Government voted to award a copy of the late Clemens Herschel's book, entitled "Frontinus and the Water Supply of the City of Rome," to William M. Bassett for his paper on "Shrinkage of Concrete in Actual Structures," presented at a meeting of the Designers Section on December 10, 1930, and published in the March, 1931, JOURNAL; also a copy each to John B. Belknap and Clarence I. Sterling, co-authors, for their paper on "Iron in the Ground Water Supplies of Massachusetts," presented at the meeting of the Sanitary Section on December 2, 1931, and printed in the January, 1932, JOURNAL. The President made the presentation of these books.

The President announced that the Board of Government had elected three honorary members, as follows: Prof. C. Frank Allen, now the senior member of the Society, having been a member since March 24, 1875, and elected honorary member March 16, 1932; Mr. Charles T. Main, a member since June 17, 1891; and elected honorary member February 17, 1932; and Dr. Karl T. Compton, President of Massachusetts Institute of Technology, elected honorary member January 27, 1932. These newly elected honorary members, except Professor Allen, who is traveling, were presented by the President with framed diplomas of membership. Previous to these presentations, Mr. Irving E. Moultrop read a biographical sketch of Mr. Charles T. Main, and Mr. Robert Spurr Weston read a biographical sketch of Dr. Compton. Each of these honorary members responded fittingly in accepting this honor.

The diplomas presented read as follows:

Boston Society of Civil Engineers

In recognition of his great attainments as an Engineer among more than half a century of busy and useful engineering service, his helpful labors as President of this Society, and the high ethical and professional standards which he has advocated and practised —

CHARLES THOMAS MAIN

has been duly elected an

Honorary Member

by the

Boston Society of Civil Engineers

By direction of the Board of Government
February 17, 1932

(SEAL) HARRY E. SAWTELL,
President.
EVERETT N. HUTCHINS,
Secretary.

Boston Society of Civil Engineers

In recognition of his scientific attainments, his achievements in research, and his notable contributions to engineering education —

KARL TAYLOR COMPTON, D.ENG.

has been duly elected an

Honorary Member

of the

Boston Society of Civil Engineers

By direction of the Board of Government
January 27, 1932

(SEAL) HARRY E. SAWTELL,
President.
EVERETT N. HUTCHINS,
Secretary.

A very pleasing musical program was presented by Mr. George Millrood, violinist, of Toledo and Boston, accompanied by Miss Ethel M. Hutchinson, who also rendered piano solos.

The President then introduced Prof. Ralph Evans Freeman, Associate Professor of Economics, Massachusetts Institute of Technology, who gave an extremely interesting address on "International Factors in the Depression."

One hundred and thirty-three members and guests attended the dinner.

The meeting adjourned at 9.30 P.M.

Everett N. Hutchins, *Secretary.*

Sanitary Section

March 2, 1932. — At the annual meeting, at 6 P.M., 22 members and guests dined at Patten's Restaurant.

At 7.15 P.M. the meeting was called to order by the Chairman, in the Societies' rooms, with 48 members and guests present. The minutes of the previous meeting, February 3, were read by the clerk and approved by the Section.

The report of the Executive Committee was read and approved.

The report of the Nominating Committee was read and accepted.

It was *Voted* that nominations be closed, and that the clerk be instructed to cast one ballot for the nominees. The ballot was cast as directed.

The following officers were elected for the coming year:

Chairman — Stuart E. Coburn.
Vice-Chairman — George G. Bogren.
Clerk — Frank L. Flood.
Executive Committee — Prof. Thomas R. Camp, Theodore F. Hatch, Paul F. Howard.

The speaker of the evening, Prof. Thomas R. Camp of Massachusetts Institute of Technology, was introduced and presented a paper entitled "The Hydraulics of Sewer Transitions." The paper was illustrated by lantern slides.

The paper described in detail the conditions existing at various stages of different types of transitions, and showed the derivation of formulæ for computing the hydraulic elements necessary in the design of hydraulic transitions.

The paper was discussed by Mr. Kennison and Professors Fair and Burden.

Adjourned at 9.30 P.M.

George G. Bogren, *Clerk.*

Designers Section

March 9, 1932. — The regular meeting of the Designers Section was called to order at 6.10 P.M. by the Chairman in the Affiliation Rooms.

The minutes of the February meeting were omitted, having previously been published.

The report of the Nominating Committee was read and it was voted that the clerk cast one ballot for those nominated. The new officers are as follows:

Chairman — Lawrence G. Ropes.
Vice-Chairman — Frederic N. Weaver.
Clerk — Herman G. Dresser.
Executive Committee — Henry Brask, William W. Bigelow, Edward A. Varney.

The Chairman then introduced Prof. F. Alexander Magoon of Massachusetts Institute of Technology, who spoke on "The Strength of Ships." Professor Magoon traced the history of shipbuilding from the earliest times to the present, and described many interesting points of design and construction that are peculiar to ships.

Many joined in the discussion following the talk.,

Twenty-two members and guests were present.

The meeting adjourned at 7.50 P.M.

Herman G. Dresser, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[April 20, 1932]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

Abel, Robert, Mattapan, Mass. (Age 21, b. Chelsea, Mass.) Student at Northeastern University, and during co-operative periods is employed by a building concern. Refers to *H. B. Alvord, C. S. Ell, J. B. Komich, W. E. Nightingale.*

Burstein, Morris Bernard, Chelsea, Mass. (Age 23, b. Kiev, Russia.) At present a senior at Tufts Engineering School. During the summer of 1930 was engineering aide in the Highway Department of Massachusetts Public Works. During summer of 1931 was inspector of construction on the new Weston by-pass. Refers to *Robinson Abbott, H. P. Burden, S. L. Conner, F. N. Weaver.*

Casagrande, Arthur, Cambridge, Mass. (Age 30, b. Haidenschaft, Italy, formerly Austria.) Studied civil engineering at the Technische Hochschule in Vienna from 1919 to 1924. During this time had some practical experience in tunnel construction in connection with the hydro-electric development of Austria, during two summers, six months in all. After graduating, was for two years assistant in hydraulics at Technische Hochschule in Vienna. Came to this country in April, 1926, and from May to November, that year, was draftsman with Carnegie Steel Company in Newark, N. J.; from November, 1926, to date employed as research assistant and research specialist with United States Bureau of Public Roads, Washington, D. C., with headquarters at Massachusetts Institute of Technology. Refers to *C. B. Breed, J. S. Crandall, I. B. Crosby, F. A. Marston, C. M. Spofford.*

Presper, Stewart Henry, Wakefield, Mass. (Age 23, b. Santa Clara, N. Y.) Graduated from Wakefield High School in 1927, and went to work for A. G. Tomasello & Son, Contractors, until January, 1929, at which time entered Northeastern University. During co-operative periods has been employed by A. G. Tomasello & Son as engineer and time keeper; as rodman for city of Medford; also with Turner Construction Company; and as rodman and material clerk in the construction of a hospital; is now working for the Donnelly Advertising Company every alternate five weeks. Refers to *H. B. Alvord, J. W. Copithorne, T. H. Fisk, J. B. Komich.*

Thomas, John Grimes Walker, Boston, Mass. (Age 30, b. Boston, Mass.) Graduate of Harvard College in 1924, and Harvard Engineering School in 1927, *cum laude.* Was assistant to Prof. L. J. Johnson during the winter of 1927–28. Started work as a statistician with Walworth Company in the fall of 1928, and kept at that until the end of 1929, when he changed to sales promotion work with same firm; then worked for a while with Turner Construction Company on the new Harvard physics laboratory until December; at intervals with Howard M. Turner, on water power and valuation work; at present is holding a temporary position with the city of Boston. Refers to *G. M. Fair, G. E. Griffin, L. J. Johnson, H. M. Turner.*

For Transfer from Grade of Junior.

Giblin, Thomas G., Wakefield, Mass. (Age 27, b. Boston, Mass.) Graduate of Mechanic Arts High School in 1923; graduate of Northeastern University in 1927, civil engineering. During co-operative periods of second, third and fourth years worked with Charles J. Elliott, civil engineer and surveyor of land surveys in Boston and surrounding cities. From May, 1927, to date employed by the Massachusetts Department of Public Works as transitman, work in office for Preliminary Department on estimates of road projects, and on construction jobs in the capacity of assistant and resident engineer. Refers to *G. A. Haskins, K. G. Hulsman, D. M. Moore, L. A. Ranlett, G. A. Scott.*

Perry, George Nelson, Waltham, Mass. (Age 27, b. Waltham, Mass.) Graduate of Northeastern University, since which has been employed by J. R. Worcester & Co. Refers to *H. B. Alvord, J. W. Howard, E. E. Pettee, J. R. Worcester, Thomas Worcester.*

ADDITIONS

Member

Edward C. Roche, Massachusetts Institute of Technology, Cambridge, Mass.

Honorary Members

Dr. Karl T. Compton, Massachusetts Institute of Technology, Cambridge, Mass.

Charles T. Main, 201 Devonshire Street, Boston, Mass.

Juniors

Louis Feldman, 10 Bassett Street, Chelsea, Mass.

Israel Gold, 643 Beach Street, Revere, Mass.

Henry R. Hamilton, 247 Kent Street, Brookline, Mass.

Gerald T. Pillsbury, 247 Kent Street, Brookline, Mass.

Frank Venti, 50 Neptune Road, East Boston, Mass.

DEATHS

Edwin H. Rogers	Mar. 6, 1932
Granville Johnson	Mar. 12, 1932
Herbert C. Daggett	Mar. 17, 1932
Elwyn L. Russell	Mar. 20, 1932
Edward S. Smilie	Mar. 21, 1932

ANNUAL REPORTS

Report of the Board of Government for the Year 1931–32

BOSTON, March 16, 1932.

To the Boston Society of Civil Engineers:

Pursuant to the requirements of the By-Laws, the Board of Government presents its report for the year ending March 16, 1932.

Membership

Forty-one new members, associates and juniors, have been added during the year, of whom 3 were former members reinstated. One honorary member has been added and two present members have been elected honorary members; twenty-three juniors have been transferred to the grade of member.

During the year 14 members have died, 28 have resigned, 74 have forfeited membership for non-payment of dues, making the total deductions 116.

The present membership of the Society consists of 4 honorary members, 791 members, 160 juniors, 17 associates, and 1 member of the Sanitary Section only, making a total membership of 973, a net loss for the year of 74.

The honorary membership list now is as follows:

John R. Freeman, elected September 19, 1917.
Charles T. Main, elected January 28, 1932.
Dr. Karl T. Compton, elected February 17, 1932.
Prof. C. Frank Allen, elected March 16, 1932.

Deaths

Lester W. Tucker, March 24, 1931.
Henry D. Woods, April 2, 1931.
Alger A. Conger, May 26, 1931.
Edward Willard Howe, June 27, 1931.
George Fillmore Swain, July 1, 1931.
Carl H. Nelson, July 23, 1931.
Frank P. Morrill, July 26, 1931.
Charles S. Parsons, October 15, 1931.
Howard A. Carson, October 26, 1931.
Ernest M. Young, November 17, 1931.
Luke D. Mullen, January 7, 1932.
Thomas F. Bowes, January 24, 1932.
Edwin H. Rogers, March 6, 1932.
Granville Johnson, March 12, 1932.

Remission of Dues

Under authority of By-Law 8, the Board of Government has remitted the dues of twenty-seven members.

Exemption from Dues

Sixty-four members are now exempt from dues in accordance with By-Law 8, which provided that "a member of any grade who has paid dues for forty years, or who has reached the age of seventy years and has paid dues for thirty years, shall be exempt from further dues."

Meetings of the Society

Eight regular meetings have been held during the year, and a joint outing with the New England Water Works Association on June 23, 1931, at the Sandy Burr Country Club, Wayland. The April, 1931, meeting was held jointly with the Engineering Societies of Boston, and one inspection trip was made.

The average attendance at the meetings of the Society was 141, the largest being 330 and the smallest 65. Buffet suppers have been a feature of all the meetings. At these meetings papers and addresses have been given, as follows:

March 19, 1931. — Annual Meeting. Address of retiring President, Lewis E. Moore, "A Suggestion for Engineering Education," followed by dinner, smoker and entertainment.

April 15, 1931. — Joint meeting with the Engineering Societies of Boston. "Design and Construction of Lighter-than-Air Craft," by V. R. Jacobs, Assistant Manager, Aeronautical Department, Goodyear Tire and Rubber Company. Members of the Boston Society of Civil Engineers and their families attended a supper in Chipman Hall previous to the lecture.

May 20, 1931. — "Fifteen-Mile Falls Lower Development," by Clarence R. Bliss, Assistant Designing Engineer, New England Power Engineering and Service Corporation.

September 23, 1931. — Inspection trip to Watertown Arsenal, Watertown, under direction of Col. Tracy C. Dickson, during the afternoon, and in the evening a talk by Colonel Dickson on "The Work of the Watertown Arsenal."

October 21, 1931. — "Traffic and Operation of New England Railroads," by Prof. J. B. Babcock, Massachusetts Institute of Technology; and "Railway Consolidation in New England," by Prof. Charles B. Breed, Massachusetts Institute of Technology.

November 18, 1931. — "Wyman Dam and Hydro-Electric Power Development at Bingham, Maine," by H. K. Fairbanks, Engineering Department, New England Public Service Company, Augusta, Maine.

December 16, 1931. — "Geological Problems of Engineering," by Prof. Kirk Bryan, Professor of Physiography, Department of Geology and Geography, and Harvard Engineering School, Harvard University.

January 28, 1932. — "The Mechanical and Hydraulic Features of the McIndoes Falls Hydro-Electric Development of the Grafton Power Company," by Harry E. Popp, Mechanical Engineer, New England Power Engineering and Service Corporation, Boston, Mass.

February 17, 1932. — Joint meeting with the Student Engineering Societies of Massachusetts Institute of Technology, Harvard University, Tufts College and Northeastern University. "Recent Developments in the Design and Construction of Highways," by Raymond W. Coburn, Construction Engineer, Massachusetts Department of Public Works; and "Recent Developments in the Use and Maintenance of Highways," by George H. Delano, Maintenance Engineer, Massachusetts Department of Public Works.

Sanitary Section Meetings

The Sanitary Section has held five meetings and one excursion during the year, with an average attendance of 34. The following papers have been presented:

March 4, 1931. — Annual Meeting. "The Trial Water Filtration Plant at Ottawa," by Dr. George G. Nasmith of Gore, Nasmith & Storrie, Consulting Engineers of Ontario, Ont.

April 1, 1931. — "Mosquito Control," by Edward Wright, Chairman, Massachusetts State Reclamation Board. Discussion by Robert W. Wales, Entomologist of the Board.

June 6, 1931. — Excursion to Woonsocket, R. I., Sewage Treatment Works, as guest of Joseph H. White, Commissioner of Public Works.

October 7, 1931. — "Financial Considerations of Municipal Sanitation," by Charles W. Sherman of Metcalf & Eddy.

December 2, 1931. — "Occurrence of Iron in Ground Water Supplies of Massachusetts," by Clarence I. Sterling and John B. Belknap, Engineering Assistant and Past Engineering Assistant, respectively, Massachusetts Department of Public Health.

February 3, 1932. — Motion Pictures of Sewage Treatment Works in England and United States, by E. Sherman Chase of Metcalf & Eddy.

Designers Section Meetings

The Designers Section has held seven regular meetings and one joint meeting with the Highway Section during the year. The average attendance has been 76. The following papers have been presented:

March 11, 1931. — Annual Meeting. "Piles and Pile Foundations," by J. Stuart Crandall of Crandall Engineering Company.

April 8, 1931. — "Light Weight Floor and Roof Construction," by Burton S. Brown and Joseph B. Harriman.

May 13, 1931. — Joint meeting with Highway Section. "The Structural Design of the New East Boston Tunnel," by Benjamin A. Loveland, Assistant Engineer, Boston Transit Department.

October 14, 1931. — "The Correction Works for the Harbor of Wilhelmshaven, Germany, and Protection Works on the Island of Wangeroog," by Prof. Kenneth C. Reynolds, Massachusetts Institute of Technology.

November 12, 1931. — "Moisture Penetration in Masonry Walls," by Howard A. Gray, Engineer, Morton C. Tuttle Company.

December 9, 1931. — Discussion of the "Proposed Boston Building Code," by Edward W. Roemer, Building Commissioner, city of Boston, Paul W. Norton, Maurice A. Reidy, and John R. Nichols.

January 13, 1932. — "Physical Properties of Clay," by Arthur Casagrande, United States Bureau of Public Roads.

February 10, 1932. — "Design of Form Work for Concrete," by Joseph Tucker of the Tucker Concrete Form Company, and Edwin F. Allbright of the Grid Flat Slab Corporation.

Highway Section Meetings

The Highway Section has held four regular meetings during the year. The average attendance has been 54. The following papers have been presented:

February 25, 1931. — Annual Meeting. "Preferred Speeds," by Clarence P. Taylor, Traffic Engineer, Massachusetts Department of Public Works.

May 13, 1931. — Joint Meeting with Designers Section. "Structural Design of the New East Boston Tunnel," by Benjamin A. Loveland, Assistant Engineer, Boston Transit Department.

September 19, 1931. — Inspection trip to Memorial Drive Underpass at Massachusetts Avenue, Cambridge, and to the new Tyngsborough Bridge across the Merrimack River at Tyngsborough.

November 24, 1931. — "Traffic Regulation and Control Methods in the Large American Cities," by D. Grant Mickle, Sheldon Traveling Fellow at Harvard University.

Northeastern University Section

The Northeastern University Section has held five meetings during the year, including inspection trips, with an average attendance of 24.

May 21, 1931. — Annual Meeting. "Design and Erection of the Tyngsborough Bridge," by A. E. Sanderson, Chief Draftsman, Boston Bridge Works, and A. E. Harding, General Superintendent, Simpson Bros. Corporation.

October 9, 1931. — Excursion to new underpass of Memorial Drive, under Massachusetts Avenue, Cambridge. Mr. McSweeney of the Metropolitan District Commission acted as guide.

November 10, 1931. — "What Comes before Construction," by Col. F. M. Gunby of Chas. T. Main, Inc.

December 10, 1931. — "Triangulation of the Cascade Tunnel in the State of Washington," by Prof. Henry B. Alvord, Northeastern University.

January 15, 1932. — Inspection trip to North Station Terminal, Boston & Maine Railroad. Mr. Baldwin of the Boston & Maine Railroad acted as guide.

Journal

The report of the Editor of the JOURNAL for the calendar year 1931 shows that there have been printed ten issues, containing a total of 515 pages, and an average of 1,438 copies of each issue was printed. The net cost in 1930 was $3,812.86, or $5.27 per page. The net cost of the JOURNAL in 1931 was $2,634.18, or $5.11 per page. The report of the Editor will be printed in the April JOURNAL.

Permanent Fund

The Society again authorized the use of the income of the Permanent Fund in payment of current expenses. Details concerning the fund will be found in the Treasurer's report.

FitzGerald Fund

The present value of the fund established by a bequest from the late Desmond FitzGerald, a past president and honorary member of the Society, is $2,317.16. The income from this fund is to be used for charitable and educational purposes. The Board voted on December 16, 1931, that the income from the FitzGerald Fund be placed at the disposal of the Welfare Committee for use as it deems best for the relief of any members of the Society within the meaning of the deed of gift. The Board voted on January 27, 1932, to appropriate this year, from the income of this fund, the sum of $100, to be known as the Boston Society of Civil Engineers Scholarship in memory of Desmond FitzGerald, to be given to a student at Northeastern University, and this will be presented to the selected student at a special meeting of the students to be held at an early date at the University.

Freeman Fund

Clifford P. Kittredge has continued for a second year, beginning June, 1931, to hold the Freeman Fund Traveling Scholarship. The scholarship provides for a year's study and work in the European Hydraulic Laboratories.

Tinkham Memorial Fund

The "Samuel E. Tinkham Fund," established at the Massachusetts Institute of Technology by the Society "to assist some worthy student of high standing to continue his studies in civil engineering," had a value of $2,715.78 on July 1, 1931. Mr. Rolf Eliassen of Belmont, Massachusetts, a student in civil engineering in the class of 1932 at Technology, has been awarded this scholarship for the year 1931–32.

Attention is called to the fact that the tuition at the Massachusetts Institute of Technology is now $500 per year, whereas the income from the Tinkham Fund is about $135. Additional donations to this fund would make it of greater service for the purpose for which it was established.

Desmond FitzGerald Medal

In accordance with the recommendation of the Committee, the Board of Government voted to award the Desmond FitzGerald Medal, for the best paper by a member of the Society published during the year, to H. K. Fairbanks, member, for his paper on "Wyman Dam and Hydro-Electric Power Development at Bingham, Maine," presented before the Society on November 18, 1931, and published in the December, 1931, JOURNAL.

Alexis H. French Fund

The Alexis H. French Fund, amounting to $1,000, was received by the Society in 1931 from the late Alexis H. French of Brookline, the income of which is to be devoted to the Library of the Society.

Section Prizes

The Board voted on April 12, 1924, "to present a prize for a worthy paper given in each Section by a member of that Section," this award to consist of books suitably inscribed.

The Board has adopted the recommendation of the Special Committee on Section Awards, and voted that the Designers Section Prize be awarded to J. Stuart Crandall, member, for his paper on "Piles and Pile Foundations," presented at the meeting of the Designers Section on March 10, 1931, and published in the May, 1931, JOURNAL.

Clemens Herschel Award

The late Clemens Herschel, a former past president and honorary member of the Society, had assigned to the Society a number of copies of his book on "Frontinus and the Water Supply of the City of Rome," with the request that the Board award one or more of the books each year as prizes for papers which have been particularly useful and commendable, and worthy of grateful acknowledgment. During the year 1931 the Society received a bequest from the late Clemens Herschel amounting to $1,000, the income of which is to be used for the same purpose.

The Board has voted to present at this time a copy of this book to the following:

1. William H. Bassett, for his paper on "Shrinkage of Concrete in Actual Structures," presented at the meeting of the Designers Section, December 19, 1930, and published in the March, 1931, JOURNAL.
2. John B. Belknap and Clarence I. Sterling, Jr., co-authors, for their paper on "Iron in the Ground Water Supplies of Massachusetts," presented at a meeting of the Sanitary Section, December 2, 1931, and published in the January, 1932, JOURNAL.

Sections

Twenty-two meetings and one inspection trip were held by the Sections of the Society during the year. These meetings of the Sections, offering opportunity for less formal discussion, have continued to demonstrate their value to their members and to the Society. The variety of subjects presented has made an appeal to the members, as indicated by the general attendance at these meetings.

Social Activities

The regular meetings of the year have been held at Chipman Hall, Tremont Temple, preceded by the buffet suppers, under the direction of the Social Activities Committee. There is general satisfaction with the efforts of the Committee on Social Activities in serving these buffet suppers and smokers. As usual, the most enthusiastic meeting was the Student Night, attended by members of the student Engineering Societies of Massachusetts Institute of Technology, Harvard, Tufts and Northeastern University, at which time representatives of the musical clubs of the latter school assisted in the entertainment.

Library

The report of the Committee on Library contains a complete account of the library activities during the year.

Society Activities

The usual special committees dealing with the general activities and conduct of the Society have included the following: Program, Publication, Library, Social Activities and Membership and Publicity. Other special committees have included the following: Subsoils of Boston, Run-off, Research, Registration and Licensing of Engineers, Relation of Sections to the Main Society, Awards of Prizes, and Welfare Committee. Each of these committees has made a distinct contribution to the Society and has developed fields of endeavor which will prove of great benefit to the members.

The Society has co-operated with the Engineering Societies of Boston in its program, and many members of the Society have served on Engineering Societies' committee.

Your Board, in conclusion, wishes to express its appreciation of the excellent work done by the officers of the Sections and by the committees of the Society.

For the Board of Government,

Harry E. Sawtell, *President.*

Report of the Treasurer

Boston, March 10, 1932.

To the Boston Society of Civil Engineers:

Your Treasurer submits the three appended tables: Table 1, presenting receipts and expenditures on account of the Society's seven funds; Table 2, recording investments and the income therefrom; and Table 3, summarizing the distribution of assets.

All interest and dividends of the Permanent Fund were transferred to the Current

Fund. Following is a brief statement of current expenditures, year by year, and shows the extent to which they have been met by the transfer of Permanent Fund income:

	1927-28	1928-29	1929-30	1930-31	1931-32
Net expenditures (except such as provided for by special appropriations)	$9,922	$9,489	$9,815	$9,443	$10,086
Members' dues received	7,323	7,635	7,794	8,028	7,045
Deficit	$2,599	$1,854	$2,021	$1,415	$3,041
Transfer of Permanent Fund income to Current Fund	$2,350	$2,404	$2,544	$2,647	$2,664

Unexpended income from the investments in funds other than the Permanent Fund has accumulated as follows:

John R. Freeman Fund	$841 54
Edmund K. Turner Fund	23 47
Desmond FitzGerald Fund	312 16
Alexis H. French Fund	40 40
Clemens Herschel Fund	42 07

All securities have been carried on the books at their cost. Recent changes in the total amounts of the Society's two principal funds are as follows:

	March 1, 1928	March 1, 1929	March 1, 1930	March 1, 1931	March 10, 1932
Permanent Fund:					
Book value	$49,349	$47,876	$48,688	$49,108	$50,081
Market value	–	–	–	53,457	46,488
John R. Freeman Fund:					
Book value	$28,437	$27,473	$27,615	$26,721	$26,722
Market value	–	–	–	31,231	23,278

The total book value of all assets is $93,991.39, of which $9,905.11 is the estimated value of the library and furniture.

Respectfully submitted,

KARL R. KENNISON, *Treasurer.*

TABLE 1. — RECEIPTS AND EXPENDITURES

[Figures in parentheses are the Secretary's account numbers]

Current Fund

Receipts

Balance March 1, 1931			$2,244 24
Transferred from Permanent Fund income			2,663 62
Meetings:			
Social activities	(13)	$357 93	
JOURNAL:			
Sale of reprints	(33)	59 22	
Advertising	(34)	1,350 00	
Sale of JOURNALS	(35)	187 90	
Library:			
Fines	(45)	11 62	
Miscellaneous:			
Badges	(51)	90 00	
Binding JOURNALS for members	(52)	40 00	
Miscellaneous receipts, including $100 for unemployment relief	(54)	109 48	
Total distributed receipts		$2,206 15	
Dues	(70)	7,045 46	
Total received from Secretary for Current Fund			9,251 61
			$14,159 47

Expenditures

Office:		
Secretary — salary and expense	(1)	$300 00
Stationery, printing and postage	(2)	482 10
Incidentals and petty cash	(3)	87 15
Insurance and Treasurer's bond	(4)	30 06
Safety deposit box	(5)	10 00
Prizes	(6)	43 95
Meetings:		
Rent of halls, etc.	(11)	227 50
Stationery, printing and postage	(12)	320 61
Social activities	(13)	695 80
Reporting	(14)	36 00
Stereopticon	(15)	59 95
Annual meeting	(16)	207 10
Sections:		
Sanitary Section	(21)	134 70
Designers Section	(22)	145 74
Highway Section	(23)	36 20
Northeastern University Section	(24)	33 95
Amount carried forward		$2,850 81

Amount brought forward		$2,850 81	
JOURNAL:			
Editor — salary and expense	(31)	478 60	
Printing and postage	(32)	3,325 90	
Reprints	(33)	59 22	
Advertising, commissions, etc.	(34)	382 05	
Library:			
Librarian — salary and expense	(41)	172 50	
Periodicals	(43)	110 75	
Binding	(44)	137 65	
Miscellaneous:			
Badges	(51)	90 00	
Binding JOURNALS for members	(52)	40 00	
Board of Reference	(53)	9 97	
Committee on unemployment relief	(54)	100 00	
Total distributed expenditures		$7,757 45	
Engineering Societies of Boston	(59)	4,535 00	
			$12,292 45
Balance March 10, 1932			1,867 02
			$14,159 47

Permanent Fund

Receipts

Balance March 1, 1931:			
Investments		$48,857 08	
Cash		251 29	
			$49,108 37
Income from investments:			
Interest:			
Bonds		$845 23	
Co-operative banks		928 40	
Savings banks		315 26	
Checking account		11 73	
Dividends		563 00	
Total interest and dividends		$2,663 62	
Profit on sales		612 50	
			3,276 12
Entrance fees	(61)		360 00
			$52,744 49

Expenditures

Income transferred to Current Fund		$2,663 62
Balance March 10, 1932:		
Investments	$49,919 29	
Cash	161 58	
		50,080 87
		$52,744 49

John R. Freeman Fund

PRINCIPAL ACCOUNT

Receipts

Balance March 1, 1931:		
Investments	$25,380 30	
Cash	499 70	$25,880 00

Expenditures

None.		
Balance March 10, 1932:		
Investments	$25,843 88	
Cash	36 12	$25,880 00

INTEREST ACCOUNT

Receipts

Balance March 1, 1931		$841 33
Interest — Checking account	$10 30	
Dividends	1,801 91	
Total interest and dividends		1,812 21
		$2,653 54

Expenditures

Traveling scholarships	$1,809 00	
Miscellaneous	3 00	
	(62)	$1,812 00
Balance March 10, 1932		841 54
		$2,653 54

Edmund K. Turner Fund

PRINCIPAL ACCOUNT

(No change during year)

Balance March 10, 1932 — Investments	$993 75

INTEREST ACCOUNT

Receipts

Balance March 1, 1931		$14 17
Interest — Bond		50 00
		$64 17

Expenditures

Books	(63)	$40 70
Balance March 10, 1932		23 47
		$64 17

Desmond FitzGerald Fund

Principal Account

(No change during year)

Balance March 10, 1932 — Investments	$2,005 00

Interest Account

Receipts

Balance March 1, 1931	$212 16
Interest — Bonds	100 00
	$312 16

Expenditures

None.

Balance March 10, 1932	$312 16

Alexis H. French Fund

Principal Account

Receipts

Balance March 1, 1931 — Investments	$1,000 00

Expenditures

None.

Balance March 10, 1932:		
Investments	$1,040 00	
Cash	—40 40	
		$1,000 00

Interest Account

Receipts

Interest — Savings bank	$40 40

Expenditures

None.

Balance March 10, 1932	$40 40

Clemens Herschel Fund

Principal Account

Receipts

Balance March 1, 1931 — Investments	$1,000 00

Expenditures

None.

Balance March 10, 1932:		
Investments	$1,042 07	
Cash	—42 07	
		$1,000 00

INTEREST ACCOUNT

Receipts

Interest — Savings bank	$42 07

Expenditures

None.

Balance March 10, 1932	$42 07

TABLE 2. — RECORD OF INVESTMENTS

PERMANENT FUND

	DURING THE YEAR MARCH 1, 1931 – MARCH 10, 1932						MARCH 10, 1932		
					SOLD OR MATURED				
	Date of Maturity or Classification	Fixed or Current Interest Rate	Interest or Dividends received	Additional Amount invested	Amount received	Profit on Sale	Par Value or Number of Shares	Market Value	Book Value
Bonds									
American Tel. & Tel. Co.	Dec. 1, 1946	5 %	$50 00	–	–	–	$1,000 00	$1,022 50	$993 75
Arkansaw Water Co.	Oct. 1, 1956	5 %	50 00	–	–		1,000 00	830 00	1,005 00
Blackstone Valley Gas & Elec. Co.	Jan. 1, 1939	5 %	100 00	–	–	–	2,000 00	1,940 00	1,995 00
Chicago & Northwestern Ry. Co.	Nov. 1, 1987	5 %	50 00	–	–	–	1,000 00	795 00	1,102 50
Illinois Power & Light Corp.	Dec. 1, 1956	5 %	28 33	$981 25	–		1,000 00	750 00	981 25
Kansas Power & Light Co.	Feb. 1, 1947	6 %	—1 33	915 00	–	–	1,000 00	900 00	915 00
Louisiana Power & Light Co.	Dec. 1, 1957	5 %	—8 61	835 00		–	1,000 00	920 00	835 00
Mississippi Power & Light Co.	Dec. 1, 1957	5 %	12 65	972 50	–	–	1,000 00	810 00	972 50
Pennsylvania Power & Light Co.	Apr. 1, 1981	4½%	18 00	970 00	–	–	1,000 00	885 00	970 00
Public Electric Light Co.	Oct. 1, 1956	5½%	75 01	975 00	–	–	2,000 00	1,360 00	1,975 00
Southern Pacific Co.	Mar. 1, 1977	4½%	20 50	955 00	–	–	1,000 00	825 00	955 00
Superior Water, Light & Power Co.	May 1, 1931	4 %	80 00	–	$4,000 00	$652 50	–	–	–
Tampa Electric Co.	Jun. 1, 1933	5 %	73 33	–	2,010 00	10 00	–	–	–
Tennessee Public Service Co.	Oct. 1, 1970	5 %	4 30	1,002 50	–	–	1,000 00	765 00	1,002 50
Union Elec. Light & Power Co.	Sep. 1, 1932	5 %	93 05	–	2,000 00	—50 00	–	–	–
Wheeling Electric Co.	May 1, 1941	5 %	200 00	–	–	–	4,000 00	3,860 00	3,845 00
Total, Bonds	–	–	$845 23	$7,606 25	$8,010 00	$612 50	–	$15,662 50	$17,547 50
Co-operative Banks									
Merchants Co-operative Bank	Dec. 1931	–	$25 80	$65 80	$1,920 90	–	10 Ser. 108		–
Merchants Co-operative Bank	Dec. 1939	5¼%	54 00	414 00	–	–	30 Ser. 140		$1,594 20
Merchants Co-operative Bank	Mat. Shs.	5¼%	52 50	–	2,000 00	–	–		–
Volunteer Co-operative Bank	July 1932	–	67 20	157 20	2,739 90	–	15 Ser. 121		–

Volunteer Co-operative Bank	Jan. 1936	5 %	140 50	440 50	–	–	25 Ser. 135		3,053 00
Volunteer Co-operative Bank	Mat. Shs.	5 %	100 00	–	–	–	$2,000 00		2,000 00
Watertown Co-operative Bank	Reinv. Ser.	5 %	488 40	488 40	420 40	–	40 Ser. 75		7,449 60
Total, Co-operative Banks	–	–	$928 40	$1,565 90	$7,081 20	–	–	$14,096 80	$14,096 80
Savings Banks									
Eliot Savings Bank, Boston	–	4½%	$151 46	$151 46	–	–	–		$3,303 34
Lynn Five Cents Savings Bank	–	5 %	163 80	163 80	–	–	–		3,572 11
Total, Savings Banks	–	–	$315 26	$315 26	–	–	–	$6,875 45	$6,875 45
Stocks									
American Tel. & Tel. Co.	–	$9 00	$414 00	–	–	–	46	$6,072 00	$5,346 04
Consolidated Gas Co. of N. Y.	–	4 00	40 00	$1,906 50	–	–	20	1,350 00	1,906 50
General Electric Co.	–	1 00	16 00	792 00	–	–	20	430 00	792 00
New England Power Assoc.	Pref.	6 00	45 00	845 00	–	–	10	520 00	845 00
New York Central R.R. Co.	–	6 00	10 00	884 25	–	–	10	320 00	884 25
North American Trust	Cum., 1953	36	18 00	765 00	–	–	200	500 00	765 00
United States Steel Co.	–	4 00	20 00	860 75	–	–	10	500 00	860 75
Total, Stocks	–	–	$563 00	$6,053 50	–	–	–	$9,692 00	$11,399 54
Cash	–	–	11 73	–	–	–	–	161 58	161 58
Total, Permanent Fund	–	–	$2,663 62	$15,540 91	$15,091 20	$612 50	–	$46,488 33	$50,080 87

TABLE 2. — RECORD OF INVESTMENTS — *Continued*

JOHN R. FREEMAN FUND

	DURING THE YEAR MARCH 1, 1931–MARCH 10, 1932						MARCH 10, 1932		
	Date of Maturity or Classification	Fixed or Current Interest Rate	Interest or Dividends received	Additional Amount invested	SOLD OR MATURED Amount received	SOLD OR MATURED Profit on Sale	Par Value or Number of Shares	Market Value	Book Value
Alabama Power Co.	Cum. Pref.	$7 00	$70 00	–	–	–	10	$880 00	$980 00
American Superpower Corp.	Pref'nce	6 00	120 00	–	–	–	20	1,210 00	1,940 00
Carolina Power & Light Co.	Cum. Pref.	7 00	70 00	–	–	–	10	830 00	1,020 00
Commonwealth & Southern Corp.	Cum. Pref.	6 00	48 00	–	–	–	8	544 00	1,019 89
Commonwealth & Southern Corp.	Com.	30	8 75	–	–	–	25	100 00	
Consumers Power Co.	Cum. Pref.	6 00	60 00	–	–	–	10	920 00	1,020 00
Electric Power & Light Corp.	Cum. Pref.	7 00	70 00	–	–	–	10	580 00	1,076 20
Electric Power & Light Corp.	Com.	1 00	6 25	$76 20	–	–	10	150 00	
Georgia Power Co.	Cum. Pref.	6 00	84 00	–	–	–	14	1,092 00	1,143 33
Minnesota Power & Light Co.	Pref.	7 00	70 00	–	–	–	10	880 00	980 00
Mississippi River Power Co.	Cum. Pref.	6 00	60 00	–	–	–	10	830 00	900 00
Montreal Light, Heat & Power Cons.	Com.	1 50	156 78	71 38	–	–	66	1,980 00	1,497 68
New England Power Assoc.	Pref.	6 00	60 00	–	–	–	10	520 00	970 00
New England Power Co.	Pref.	6 00	60 00	–	–	–	10	910 00	960 00
Northwestern Electric Co.	Cum. 1st Pr.	7 00	70 00	–	–	–	10	780 00	980 00
Great West. Power Co. of Calif.	Cum. Pref.	–	17 50	–	$990 00*	–	–	–	–
Pacific Gas & Electric Co.	Cum. 1st Pr. 6%	1 50	85 00	990 00*	–	–	60	1,440 00	1,922 02
Pacific Gas & Electric Co.	Pref. 5½%	1 375	22 93		–	–	20	450 00	
Pacific Gas & Electric Co.	Com.	2 00	125 70	150 00	–	–	64	2,304 00	1,808 79
Penn. Gas & Elec. Corp.	Pref.	7 00	210 00	–	–	–	30	975 00	2,955 00
Southern Calif. Edison Co.	Cum. Orig. Pr.	2 00	80 00	–	–	–	40	1,600 00	1,161 22
Southern Calif. Edison Co.	Com.	2 00	37 00	166 00	–	–	20	650 00	539 75
The Tenn. Elec. Power Co.	1st Pref.	7 00	70 00	–	–	–	10	910 00	950 00

Utah Power & Light Co.	Cum. Pref.	7 00	70 00	–	–	–	10	790 00	990 00
West Pennsylvania Power Co.	Cum. Pref.	7 00	70 00	–	–	–	10	1,075 00	1,030 00
Total, Stocks	–	–	$1,801 91	$463 58	–	–	–	$22,400 00	$25,843 88
Cash:									
Principal account	–	–	–	–	–	–	–	36 12	36 12
Interest account	–	–	10 30	–	–	–	–	841 54	841 54
Total, John R. Freeman Fund	–	–	$1,812 21	$463 58	–	–	–	$23,277 66	$26,721 54

EDMUND K. TURNER FUND

Bonds									
American Tel. & Tel. Co.	Dec. 1, 1946	5 %	$50 00	–	–	–	$1,000 00	$1,022 50	$993 75
Total, Bonds	–	–	$50 00	–	–	–	–	$1,022 50	$993 75
Cash — Interest account	–	–	–	–	–	–	–	23 47	23 47
Total, Edmund K. Turner Fund	–	–	$50 00	–	–	–	–	$1,045 97	$1,017 22

DESMOND FITZGERALD FUND

Bonds									
Central Power & Light Co.	Aug. 1, 1956	5 %	$50 00	–	–	–	$1,000 00	$640 00	$1,005 00
Southwestern Gas & Elec. Co.	Jan. 1, 1957	5 %	50 00	–	–	–	1,000 00	780 00	1,000 00
Total, Bonds	–	–	$100 00	–	–	–	–	$1,420 00	$2,005 00
Cash — Interest account	–	–	–	–	–	–	–	312 16	312 16
Total, Desmond FitzGerald Fund	–	–	$100 00	–	–	–	–	$1,732 16	$2,317 16

* Exchanged.

Table 2. — Record of Investments — *Concluded*

Alexis H. French Fund

	During the Year March 1, 1931 – March 10, 1932						March 10, 1932		
	Date of Maturity or Classification	Fixed or Current Interest Rate	Interest or Dividends received	Additional Amount invested	Sold or Matured		Par Value or Number of Shares	Market Value	Book Value
					Amount received	Profit on Sale			
Savings Banks									
Boston Five Cents Savings Bank	–	4 %	$40 40	$40 40	–	–	–	–	$1,040 40
Total, Savings Bank	–	–	$40 40	$40 40	–	–	–	$1,040 40	$1,040 40
Cash:									
Principal account	–	–	–	–	–	–	–	−40 40	−40 40
Interest account	–	–	–	–	–	–	–	40 40	40 40
Total, Alexis H. French Fund	–	–	$40 40	$40 40	–	–	–	$1,040 40	$1,040 40

Clemens Herschel Fund

	Date of Maturity or Classification	Fixed or Current Interest Rate	Interest or Dividends received	Additional Amount invested	Amount received	Profit on Sale	Par Value or Number of Shares	Market Value	Book Value
Savings Banks									
Charlestown Five Cents Savings Bank	–	5 %	$42 07	$42 07	–	–	–	–	$1,042 07
Total, Savings Bank	–	–	$42 07	$42 07	–	–	–	$1,042 07	$1,042 07
Cash:									
Principal account	–	–	–	–	–	–	–	−42 07	−42 07
Interest account	–	–	–	–	–	–	–	42 07	42 07
Total, Clemens Herschel Fund	–	–	$42 07	$42 07	–	–	–	$1,042 07	$1,042 07

ALL FUNDS

Total, Investment Funds	–	–	$4,708 30	$16,086 96	$15,091 20	$612 50	–	$74,626 59	$82,219 26
Cash in Current Fund	–	–	–	–	–	–	–	1,867 02	1,867 02
Total, Funds	–	–	–	–	–	–	–	$76,493 61	$84,086 28

TABLE 3. — DISTRIBUTION OF ASSETS, MARCH 10, 1932

	Current Fund	INVESTMENTS						Inventory	Total
		Permanent Fund	John R. Freeman Fund	Edmund K. Turner Fund	Desmond FitzGerald Fund	Alexis H. French Fund	Clemens Herschel Fund		
Assets									
Bonds	–	$17,547 50	–	$993 75	$2,005 00	–	–	–	$20,546 25
Co-operative banks	–	14,096 80	–	–	–	–	–	–	14,096 80
Savings banks	–	6,875 45	–	–	–	$1,040 40	$1,042 07	–	8,957 92
Stocks	–	11,399 54	$25,843 88	–	–	–	–	–	37,243 42
Cash:									
Principal account	–	161 58	36 12	–	–	–40 40	–42 07	–	115 23
Interest account	$1,867 02	–	841 54	23 47	312 16	40 40	42 07	–	3,126 66
Total Funds — Book value	$1,867 02	$50,080 87	$26,721 54	$1,017 22	$2,317 16	$1,040 40	$1,042 07	–	$84,086 28
(Market value)	(1,867 02)	(46,488 33)	(23,277 66)	(1,045 97)	(1,732 16)	(1,040 40)	(1,042 07)	–	(76,493 61)
Library and furniture	–	–	–	–	–	–	–	$9,905 11	9,905 11
Total Assets — Book value	–	–	–	–	–	–	–	–	$93,991 39
Liabilities									
None.									
Corresponding Totals in Previous Years									
Book value:									
March 1, 1931	$2,244 24	$49,108 37	$26,721 33	$1,007 92	$2,217 16	$1,000 00	$1,000 00	$9,905 11	$93,204 13
(Market value)	–	(53,457 08)	(31,230 83)	–	–	–	–	–	–
March 1, 1930	1,011 64	48,688 37	27,614 81	1,019 37	2,217 16	–	–	9,905 11	90,456 46
March 1, 1929	489 08	47,875 87	27,472 79	1,018 92	2,117 16	–	–	9,905 11	88,878 93
March 1, 1928	60 79	49,348 76	28,437 07	1,042 10	2,017 16	–	–	9,905 11	90,689 41

Report of Auditing Committee for Year 1931–32

Boston, March 12, 1932.

We have examined the accounts of the treasurer of the Boston Society of Civil Engineers and find the books properly kept and the various expenditures of the past year supported by duly approved receipts.

The Treasurer has also accounted to us for the investments and cash on hand, as submitted in the above report.

John B. Babcock, 3d,
Gordon M. Fair,
Auditing Committee of the Directors of the
Boston Society of Civil Engineers.

Report of the Secretary

Boston, Mass., March 16, 1932.

Everett N. Hutchins, Secretary, *in Account with the* Boston Society of Civil Engineers, *Dr.*

For cash received during the year ending March 16, 1932:

From entrance fees, new members and transfers:			
11 new members and associates at	$10 00	$110 00	
27 juniors at	5 00	135 00	
23 juniors transferred to members at	5 00	115 00	
Total from entrance fees			$360 00
From annual dues for 1931–32, including dues from new members		$6,869 46	
From back dues		136 00	
From dues for 1932–33		40 00	
Total from dues			7,045 46
From advertisements			1,350 00
From sale of Journals			187 90
From reprints			59 22
From library fines and miscellaneous			21 10
From sale of badges			90 00
From binding Journals for members			40 00
From Social Activities Committee (smokers)			357 93
From donation for Welfare work			100 00
Total receipts			$9,611 61

The above amount has been paid to the Treasurer, whose receipts the Secretary holds.

The appended table shows the estimated and the actual expenditures for the year.

Everett N. Hutchins, *Secretary.*

Comparison of Estimated and Actual Expenditures, 1931–32

	Account Number	Estimated Expenditures		Actual Expenditures	
Office:					
Secretary, salary and expense	(1)	$325 00		$300 00	
Stationery, printing and postage	(2)	300 00		482 10	
Incidentals and petty cash	(3)	125 00		87 15	
Insurance and Treasurer's bond	(4)	35 00		30 06	
Safety deposit box	(5)	10 00		10 00	
Prizes	(6)	40 00		43 95	
Total			$835 00		$953 26
Meetings:					
Rent of halls, etc.	(11)	$250 00		$227 50	
Stationery, printing and postage	(12)	325 00		320 61	
Social activities (net)	(13)	400 00		337 87	
Reporting and stereopticon	(14, 15)	100 00		95 95	
Annual meeting (1931)	(16)	175 00		207 10	
Total			1,250 00		1,189 03
Sections:					
Sanitary Section	(21)	$175 00		$134 70	
Designers Section	(22)	100 00		145 74	
Highway Section	(23)	50 00		36 20	
Northeastern University Section	(24)	50 00		33 95	
Total			375 00		350 59
Journal:					
Editor, salary and expense	(31)	$500 00		$478 60	
Printing and postage	(32)	4,000 00		3,325 90	
Reprints (net)	(33)	75 00		–	
Total			4,575 00		3,804 50
Library:					
Librarian, salary and expense	(41)	$150 00		$172 50	
Books	(42)	50 00		–	
Amounts carried forward	–	$200 00	$7,035 00	$172 50	$6,297 38

Comparison of Estimated and Actual Expenditures, 1931–32 — *Concluded*

	Account Number	Estimated Expenditures		Actual Expenditures	
Amounts brought forward	–	$200 00	$7,035 00	$172 50	$6,297 38
Periodicals	(43)	160 00		110 75	
Binding	(44)	150 00		137 65	
Total			510 00		420 90
Dues to Engineering Societies of Boston	(59)		4,535 00		4,535 00
Board of Reference	(53)		50 00		9 97
Total Expenditures			$12,130 00		$11,263 25

Report of Auditing Committee for Year 1931–32

Boston, March 12, 1932.

We have examined the records and accounts of the Secretary of the Boston Society of Civil Engineers and have satisfied ourselves that they are properly kept, and that the money received has been transferred to the Treasurer.

John B. Babcock, 3d,
Gordon M. Fair,
Auditing Committee of the Directors of the
Boston Society of Civil Engineers.

Report of the Editor

BOSTON, February 1, 1932.

To the Board of Government, Boston Society of Civil Engineers:

The Editor submits the following report for the calendar year 1931 (Volume XVIII) of the JOURNAL.

During the year 1931 there have been published twelve papers presented at meetings of the Society and Sections; six other articles; one book review; two memoirs of deceased members; and the report of the Committee on Subsoils of Boston.

The ten issues of the JOURNAL contained 515 pages; an average of 1,438 copies of each issue are printed. The net cost was $2,634.18 as compared with $3,812.86 for the preceding year.

Table 1 includes a comparison of costs for the JOURNAL from 1927–31, inclusive. In Table 2 details of cost for Volume XVIII of the JOURNAL for the calendar year 1931 are shown.

The December issue of each volume contains the Contents and Index for the year.

Respectfully submitted,

EVERETT N. HUTCHINS, *Editor.*

TABLE 1. — COMPARISON OF COSTS OF JOURNAL, 1927–31, INCLUSIVE

YEAR	Vol.	Number of Pages*	GROSS COST†		NET COST‡		Number of Cuts and Inserts
			Total	Per Page	Total	Per Page	
1927 . . .	XIV	700	$4,155 56	$5 94	$2,905 31	$4 15	121
1928 . . .	XV	638	4,143 38	6 49	2,554 11	4 00	150
1929 . . .	XVI	641	4,525 79	7 06	2,854 17	4 45	116
1930 . . .	XVII	723	5,107 16	7 06	3,812 86	5 27	167
1931 . . .	XVIII	515	3,916 95	7 60	2,634 18	5 11	99

* Includes Advertising Section and Annual Index.

† Gross cost included editing, printing, mailing and reprints.

‡ Net cost equals gross cost less amounts received for advertising, subscriptions and sale of JOURNALS and reprints.

TABLE 2. — 1931 JOURNAL, VOLUME XVIII

MONTH	PAGES		CUTS	COST						
	Papers and Proceedings	Advertising	Number	Composition and Printing	Cuts	Reprints	Wrapping, Mailing and Postage	Editing and Expense	Copyright	
January	25	14	–	$180 77	$419 41	–				
February	40	15	8	201 34	45 91	–				
March	27	15	10	160 52	38 93	–				
April	57	15	8	327 21	33 47	–				
May	41	14	12	234 25	72 98	–				
June	47	13	24	240 30	68 74	$59 22				
September	44	13	9	623 85	137 52	–				
October	20	11	6	151 71	22 37	–				
November	26	11	5	133 86	24 26	–				
December	57	10	17	249 24	84 36	–				
Totals	384*	131†	99	$2,503 05	$547 95	$59 22	$328 73	$458 00	$20 00	$3,916 95

* Includes 9 pages of Annual Index.

† Includes Advertisers' Index and any other unpaid space.

Gross cost of JOURNAL		$3,916 95
Receipts from subscriptions, sale of JOURNALS and reprints	$274 92	
Receipts from advertising (net)	1,007 85	
Total receipts		1,282 77
Net cost of JOURNAL		$2,634 18

Report of the Library Committee, 1931–32

Boston, March 16, 1932.

To the Boston Society of Civil Engineers:

The Chairman submits, for the Library Committee, the following report for the year 1931–32:

During the year 291 books have been loaned to members of the Society, or to members of other societies included in the Engineering Societies of Boston, and fines amounting to $11.62 have been collected.

The books listed in last year's report as having been received from the library of Clemens Herschel have been cataloged and placed in a separate collection on the shelves of the Library.

Several shelves of books have been rearranged under the direction of Edward S. Averell. He has also checked our indexes with the books on the shelves, and has made a list of lost books. At times the taking of books from the Library without permission or record presents a serious problem, which at present has not been satisfactorily settled. Cross indexing has also been brought up-to-date.

The Committee has used the funds at its disposal in the purchase of new books, the list of which is appended to this report.

Respectfully submitted,

Henry B. Alvord,
Librarian and Chairman,
Library Committee.

Books added to the Library, 1931–32

Earthquake Damage and Insurance. J. R. Freeman. (Gift.)
Underpinning. Prentiss and White.
Car Builders Cyclopedia.
Highway Economics. Johannesson. (Gift.)
Technical Dictionary. Vol. I. Kittredge.
Principles of City Planning. Lohmann. (Gift.)
Elastic Arch Bridges. McCullock and Thayer. (Gift.)
Design of Dams. Hanna and Kennedy.
Modern Framed Structures. Vol. III. Johnson, Bryan & Turneaure.
Foundations of Bridges and Buildings. Jacoby and Davis.
Design of Steel Structures. Urquhart and O'Rourke.
Handbook of Formulas and Tables for Engineers. Pierce, Carver and O'Rourke.
Architects' and Builders' Handbook. Kidder and Parker.
Bibliography of Bibliographies in Electrical Engineering, 1918–1929.

Report of the Committee on Membership and Publicity

Boston, March 12, 1932.

To the Boston Society of Civil Engineers:

The report of the Committee on Membership and Publicity of the Boston Society of Civil Engineers for the year 1931–32 is as follows:

Membership

In view of the times, it has seemed inadvisable to attempt any organized activity toward increasing membership by high pressure methods. Many members find themselves confronted with difficulty in meeting payments for dues, and the Board can do much to relieve this situation by a continuation of its policy of granting extension of time for the payment of dues in worthy cases.

The attention of the Society has been called to the essential requirement for a normal amount of new blood in the Society by mailing to each member with one issue of the Journal during the year an application blank form in the hope that it might be handed on to a potential candidate. The Committee has also prepared and published in the January issue of the Journal an outline of the activities and purposes of the Society, condensing for ready reference in a form not hitherto available the material which is required in approaching a prospect for membership. These data in separate pamphlet form should be of material help to later Committees who may see fit to conduct intensive membership drives when better times return.

Publicity

Efforts have been made to gain publicity for the Society through notices in the press. These have met with indifferent success, due partly to the lack of editorial interest in the rather technical nature of the subjects of the meetings, and more especially to the difficulty which the amateur experiences in getting his material before the proper newspaper official in a way to get it routed elsewhere than to the waste basket. These experiences have pointed clearly to the desirability of having publicity handled by a professional who, with knowledge of news values and entrée to the newspaper offices, could bring the activities of the Society more prominently to public attention. Obviously, the present is not the time to add to the expenses of the Society for this purpose.

Respectfully submitted,

For the Committee on Membership and Publicity,

Arthur L. Shaw, *Chairman.*

Report of the Committee on Registration and Licensing of Engineers

BOSTON, March 16, 1932.

To the Boston Society of Civil Engineers:

While the members of the Committee have discussed informally the subject of registration and licensing of engineers in Massachusetts, no definite action has been taken, and it has not seemed advisable or necessary to take up these matters actively during the past year.

Respectfully submitted,

FRANK A. MARSTON, *Chairman.*

Report of the Committee on Social Activities

BOSTON, March 16, 1932.

To the Boston Society of Civil Engineers:

The Committee on Social Activities submits the following report for the year 1931–32:

At seven out of eight meetings held during the past year the Committee has served approximately 780 members and guests of the Society with buffet suppers, an average of 111 per meeting.

The largest number served at any meeting was 298, the occasion being the annual Student Night, when members of the student engineering societies of Tufts College, Northeastern University, Harvard and Massachusetts Institute of Technology were guests of the Society. The smallest number served was 45 at the September meeting.

As the April meeting was held jointly with the Engineering Societies of Boston, we decided to omit the usual buffet supper and have in its stead a catered dinner served to members of the Society and their ladies, which proved to be a very enjoyable occasion.

In June, as heretofore, an outing was held jointly with the New England Water Works Association, this year at the Sandy Burr Country Club in Wayland. About 230 members and guests of the two organizations attended, and to all appearances they enjoyed it in spite of the inclement weather.

Respectfully submitted,

RICHARD W. SHERMAN, *Chairman.*

Report of the Committee on Subsoils of Boston

BOSTON, March 16, 1932.

To the Boston Society of Civil Engineers:

The Subsoils Committee has to report that a Progress Report was printed in the September (1931) JOURNAL of last year, and that since that time no additional information has been obtained of sufficient moment to make an additional report at this time.

HARRY E. SAWTELL, *Chairman.*

Report of the Committee on Research

BOSTON, March 16, 1932.

To the Boston Society of Civil Engineers:

A report was submitted on January 27, 1932, upon subjects for research, and that report has been published in the February, 1932, JOURNAL. It is hoped that some discussion will be submitted, and that the publishing of the report may bring the subjects to the attention of members who will interest themselves in making some of the special studies suggested and in preparing papers for the Society.

The next step will be to bring certain subjects to the attention of members of the Society well qualified to discuss them.

The list of subjects is not complete, and without doubt many other worth-while subjects will be suggested after additional thought is given to the matter.

Respectfully submitted,

For the Committee,

FRANK A. MARSTON, *Chairman.*

Report of the Committee on Run-off

LOWELL, MASS., March 3, 1932.

To the Boston Society of Civil Engineers:

The Committee on Run-off has organized, and appointed a steering committee consisting of Messrs. Uhl, Kinnison, Kent, Henderson and Noyes; it will collect and study the data and be ready to report by another year, provided the drought is at an end.

The year 1931, while not a year of extreme drought like 1930, has had enough low spots to warrant including it with 1930, and 1932 may behave as badly. For this reason the work of the committee has not been stressed beyond getting the records.

The year 1930 was definitely a year of drought; 1931 a more nearly normal year, but with deficiencies of rainfall in southern New England and excess of rainfall in northern New England.

The United States Department of Agriculture in the annual Weather Bureau report for 1931 gives the following:

> There was an excess in precipitation in March, May, June, July and August, with the greatest excess in June. There was a deficiency in precipitation in January, February, April, September, October, November, with the greatest deficiency in November.

The year 1931 should be an interesting one to study.

For the Committee on Run-off,

ARTHUR T. SAFFORD, *Chairman.*

Report of the Committee on Welfare

BOSTON, March 16, 1932.

To the Boston Society of Civil Engineers:

The Welfare Committee respectfully submits herewith a report of its activities for the past year.

The Committee was appointed by President Sawtell upon authorization by the Board of Government at its September meeting, in response to a communication from the President of the Boston Society of Architects asking the Boston Society of Civil Engineers to appoint a committee which might co-operate with the Architects' Committee on Unemployment.

The Committee interpreted its duties to be of an emergency nature, and immediately undertook a thorough survey of the unemployment situation among our entire membership, to determine the extent of unemployment and the seriousness of individual cases. Efforts also were made to find engineering work for the unemployed, particularly for those whose financial situation was critical. These efforts were not successful.

At the same time the Committee, in co-operation with the Architects' Committee, outlined a scheme for creating emergency employment, which would make use of the unemployed engineers on work where their technical training and experience would be of value. This scheme proposed that funds should be solicited from the employed members of the engineering profession and used to employ the unemployed members on technical work, which otherwise could not be undertaken.

Upon consideration of the proposed scheme by the Board of Government this matter appeared to be of such importance and magnitude that the Board voted to refer it to the Engineering Societies of Boston for action. Favorable action was readily secured from the Engineering Societies of Boston, and through the enthusiastic and whole-hearted support of the member societies the Emergency Planning and Research Bureau, Inc., has now become a reality. Employees of the Bureau are divided into two groups, namely, engineers and architects. Each group is paid from funds subscribed by the members of its respective profession. Funds to the extent of over $73,000 have been pledged to carry on the work of the engineers' group, and to date approximately 140 members of the engineering profession, who were urgently in need of employment, have been put to work with the Bureau.

The members of the Welfare Committee are serving with representatives of the other member societies of the Engineering Societies of Boston as a Registration Committee for the Emergency Planning and Research Bureau, Inc. The Welfare Committee is also serving as a subcommittee to investigate and pass upon critical cases among our own membership, and is otherwise continuing its duties to our own needy members.

Recently the Boston Society of Civil Engineers has received a substantial contribution from the "Municipal Technical Engineers' Association of Boston," to be used for the express purpose of relief of our members who are out of work and need assistance. Their communication indicated that they were heartily in sympathy with our efforts to assist our unemployed members. This contribution is being expended to employ needy Society members upon work which will improve the library and records of the Society.

Respectfully submitted,

For the Welfare Committee,

Ralph W. Horne, *Chairman.*

Report of the Committee on Relation of Sections to the Main Society

Boston, Mass., March 16, 1932.

To the Boston Society of Civil Engineers:

This Committee was appointed to provide closer contact between the main Society and its four Sections which now have a membership of nearly 500 and hold twenty or more meetings each year. The following recommendations, based on a study of the activities of the Sections, are presented as a progress report with the suggestion that a similar committee be appointed next year to co-operate further with the Sections.

1. A tentative "Consolidation of Section By-Laws" to replace the four individual sets of By-Laws, which are nearly identical, has been prepared for submission to the Executive Committees of the Sections. Its adoption would result in a simplification which would be of advantage to the Sections; it would also be desirable from the standpoint of saving in cost and space in the publication of the Society's Year Book.

2. *Co-ordination of Programs.* — It is recommended that the Program Committee of the Society hold a joint meeting with the Executive Committees of all the Sections early in the new year for discussion of the programs of the Society and of the Sections, and for possible arrangement of joint meetings on subjects of particular interest to several groups. During the year, as subsequent meetings are arranged, the Committee on Relations of Sections to the Main Society could assist in the co-ordination of programs.

3. *Notices of Meetings.* — A calendar should be published each month in the JOURNAL giving the dates of all regular meetings of the Society and of the Sections to be held during the remainder of the year. In addition to the individual post-card notices now sent to the Section members in advance of a Section meeting, each notice of a meeting of the Main Society should carry a brief announcement of the dates and subjects of Section meetings for the ensuing month, with a reminder that all Society members are welcome at Section meetings.

4. *Membership and Publicity.* — All members of the Society are eligible for membership in the Sections except in the case of the Northeastern University Section, in which the membership is limited to students, graduates and members of the faculty at

that university. This fact should be given more publicity, particularly for the benefit of the new members, in order that the membership of the Sections may be increased. A proposed grade of "Student" member of the Society has been approved by the Board of Government and will be submitted to the membership for consideration at an early date. The adoption of this grade would result in a desirable influx of students of civil engineering in near-by technical schools, and would increase the interest in Section activities, particularly in the case of the Northeastern University Section.

5. *Section Prize Papers.* — The presentation of papers at Section meetings by members of the Section should be further encouraged and definite meetings set aside for this purpose. The members of the Executive Committees of the Sections are urged to give this matter special attention, and to suggest subjects to those members who are particularly qualified to present them. The list of nearly one hundred subjects recently suggested by the Society's Committee on Research (published in the February, 1932, JOURNAL) provides a wealth of material on which papers might be presented. One of the most important features of Section meetings is the informal discussion of engineering topics, particularly by the younger members. Section prizes are awarded for worthy Section papers by members, and greater effort should be made to arrange programs which will include such papers.

6. *Attendance at Section Meetings.* — Members of this co-operating Committee and the vice-presidents of the Society should attend Section meetings frequently to familiarize themselves with the work of the Sections and to show the interest of the Society in its Sections. Vice-chairmen of the Sections are urged to attend some of the meetings of the other Sections to keep in touch with the activities of all the Sections.

These recommendations have been approved by the chairmen of the Sections for 1931–32, and by the incoming chairmen for the new year. Your Committee believes that the carrying out of these recommendations will result in more effective work by the Sections, and will bring the Society and the Sections into closer contact.

Respectfully submitted,

F. H. KINGSBURY, *Chairman*, Sanitary Section.
ERWIN HARSCH, *Chairman*, Designers Section.
A. B. EDWARDS, *Chairman*, Highway Section.
J. B. KOMICH, *Chairman*, Northeastern University Section.
H. K. BARROWS. R. W. HORNE.
A. W. DEAN. J. B. BABCOCK, *Chairman*.

Report of the Executive Committee of the Sanitary Section

BOSTON, March 2, 1932.

To the Sanitary Section, Boston Society of Civil Engineers:

There have been five meetings and one excursion held during the year, as follows:

March 4, 1931. — Annual meeting. "The Trial Water Filtration Plant at Ottawa," by Dr. George G. Nasmith of Gore, Nasmith, & Storrie, Consulting Engineers of Toronto, Ont. (illustrated); *attendance*, 48.

April 1, 1931. — Special meeting. "Mosquito Control," by Edward Wright, Chairman, Massachusetts State Reclamation Board. Discussion by Robert W. Wales, Entomologist of the Board (illustrated); *attendance*, 34.

June 6, 1931. — Excursion, to Woonsocket, R. I., Sewage Treatment Works, as guests of Joseph H. White, Commissioner of Public Works; *attendance*, 35.

October 7, 1931. — Regular meeting. "Financial Considerations of Municipal Sanitation," by Charles W. Sherman of Metcalf & Eddy; *attendance*, 25.

December 2, 1931. — Regular meeting. "Occurrence of Iron in Ground Waters of Massachusetts," by Clarence I. Sterling and John B. Belknap, engineering assistant and past engineering assistant, respectively, Massachusetts Department of Public Health; *attendance*, 21.

February 3, 1932. — Special meeting. Motion Pictures of Sewage Treatment Works in England and the United States, by E. Sherman Chase of Metcalf & Eddy; *attendance*, 40.

Attendance. — Attendance at meetings has varied from 21 to 48, averaging 34. The practice of meeting to dine prior to the meetings has been continued, the attendance varying from 15 to 28, averaging 22.

Membership. — The present membership of the Section is 187, a loss of 4 since last year.

Seven new members have been added to the roll of the Section, while 11 have been lost, — 5 by resignation, 2 by forfeiture, and 4 by death. The names of the members taken from us by death are: T. F. Bowes, E. W. Howes, H. A. Symonds and E. M. Young.

Executive Committee Meetings. — Four meetings of the Executive Committee have been held during the year.

Respectfully submitted,

For the Executive Committee,

George G. Bogren, *Clerk.*

Report of the Executive Committee of the Designers Section

Boston, March 9, 1932.

To the Designers Section, Boston Society of Civil Engineers:

During the past year seven regular meetings and one joint meeting with the Highway Section have been held, as follows:

March 11, 1931. — Annual meeting. *Subject*, "Resistance of Piles;" *speaker*, Mr. J. Stuart Crandall; *attendance*, 95.

April 8, 1931. — *Subject*, "Light Weight Floor and Roof Construction;" *speakers*, Mr. Burtis S. Brown and Mr. Joseph B. Harriman; *attendance*, 65.

May 13, 1931. — *Subject*, "The Structural Design of the New East Boston Tunnel;" *speaker*, Mr. Benjamin A. Loveland, Assistant Engineer of the Boston Transit Department; *attendance*, 108. This was a joint meeting with the Highway Section.

October 14, 1931. — *Subject*, "The Correction Works for the Harbor of Wilhelmshaven, Germany, and the Protection Works on the Island of Wangeroog;" *speakers*, Prof. Kenneth C. Reynolds of Massachusetts Institute of Technology, and John R. Freeman Scholar of the Boston Society of Civil Engineers, 1927–29; *attendance*, 13.

November 12, 1931. — *Subject*, "Moisture Penetration in Masonry Walls;" *speaker*, Mr. Howard A. Gray, Engineer for the Morton C. Tuttle Company; *attendance*, 65.

December 9, 1931. — *Subject*, "Discussion of the Proposed Boston Building Code;" *speakers*, Mr. Edward W. Roemer, Building Commissioner of Boston, Mr. Paul W. Norton, Mr. Maurice A. Reidy, and Mr. John R. Nichols; *attendance*, 135.

January 13, 1932. — *Subject,* "Physical Properties of Clay;" *speaker,* Mr. Arthur Casagrande of the United States Bureau of Public Roads; *attendance,* 62.

February 10, 1932. — *Subject,* "Design of Formwork for Concrete;" *speakers,* Mr. Joseph Tucker of the Tucker Concrete Form Company, and Mr. Edwin F. Allbright of the Grid Flat Slab Corporation; *attendance,* 65.

The average attendance was 76; last year it was 34.

During the year two new members were added, seven resigned, three were dropped and two died. The net membership this year is 163; last year it was 173.

Respectfully submitted,

For the Executive Committee,

Herman G. Dresser, *Clerk.*

Report of the Executive Committee of the Highway Section

Boston, February 23, 1932.

To the Highway Section, Boston Society of Civil Engineers:

During the past year four regular meetings of the Highway Section have been held. These meetings are summarized as follows:

February 25, 1931. — Annual meeting. Election of officers, as follows: Chairman, A. B. Edwards; Vice-Chairman, Edwin H. Rogers; Clerk, Arthur P. Rice; Executive Committee, J. B. Babcock, Ernest Mathers, Francis T. McAvoy. *Speaker,* Mr. Clarence P. Taylor, Traffic Engineer of the Massachusetts Department of Public Works. *Subject,* "Preferred Speeds;" *attendance,* 28.

May 13, 1931. — *Speaker,* Mr. Benjamin A. Loveland, Assistant Engineer of the Boston Transit Department. *Subject,* "Structural Design of the New East Boston Tunnel;" *attendance,* 108. This was a joint meeting with the Designers Section.

September 19, 1931. — Inspection trip to Memorial Drive underpass at Massachusetts Avenue, Cambridge, and to the new Tyngsborough bridge across the Merrimack River at Tyngsborough; *attendance,* 16.

November 24, 1931. — *Speaker,* Mr. D. Grant Mickle, Sheldon Traveling Fellow at Harvard University. *Subject,* "Traffic Regulation and Control Methods in the Large American cities;" *attendance,* 39.

During the year the Executive Committee held two meetings.

Membership. — The present membership of the Highway Section is 90; during the year 1 new member has been enrolled; 1 has forfeited membership; and 3 have resigned.

Attendance. — The attendance at meetings during the year has ranged from 16 to 108, and has averaged 54 per meeting.

Respectfully submitted,

For the Executive Committee,

Arthur P. Rice, *Clerk.*

Report of the Executive Committee of the Northeastern University Section

MARCH 9, 1932.

To the Northeastern University Section, Boston Society of Civil Engineers:

The Northeastern University Section of the Boston Society of Civil Engineers has held five meetings, including inspection trips, during the year, as follows:

May 21, 1931. — Annual meeting. "Design and Erection of the Tyngsborough Bridge," by A. E. Sanderson, Chief Draftsman, Boston Bridge Works, and A. E. Harding, General Superintendent, Simpson Bros. Corporation.

October 9, 1931. — Excursion to new underpass of Memorial Drive under Massachusetts Avenue, Cambridge. Mr. McSweeney, Metropolitan District Commission, acted as guide.

November 10, 1931. — "What Comes before Construction," by Col. Frank M. Gunby of Chas. T. Main, Inc.

December 10, 1931. — "Triangulation of the Cascade Tunnel in the State of Washington," by Prof. Henry B. Alvord.

January 15, 1932. — Inspection trip to the North Station Terminal, Boston & Maine Railroad. Mr. Baldwin of the Boston & Maine Railroad acted as guide.

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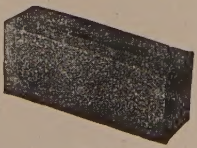

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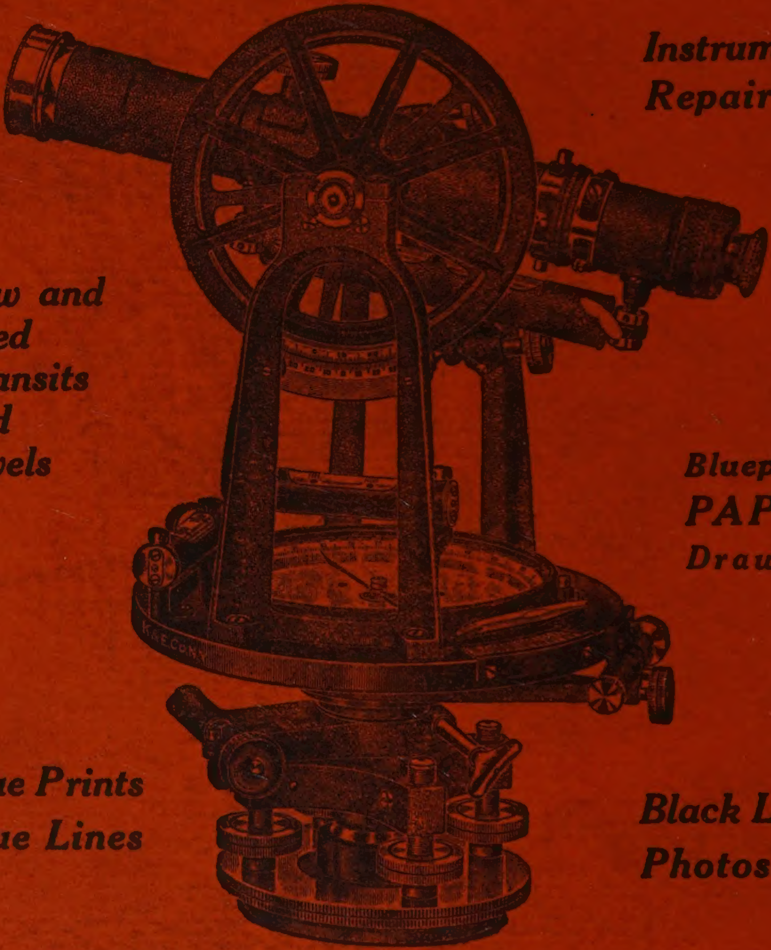

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